بسم الله الرحمن الرحيم

سیستم های مدرن جذب انرژی در

سازه های فولادی

- قابهای خمشی فولادی با دیوارهای برشی فولادی

- دیوار برشی فولادی، سیستمی است که از ورقهای فولادی، ستونهای حایل وتیرهایی که در تراز هر طبقه وجود دارند تشکیل شده است.
- رفتار دیوارهای برشی فولادی را می توان با عملکرد تیر ورق های شبیه سازی کرد.
- ستونهای قاب با دیوار برشی مانند بال های تیر ورق، تیر های آن مشابه با تقویت کننده های میانی تیر ورق و ورق فولادی به مثابه جان تیر ورق می باشند.
- ***ورق فولادی عمودی که به تیرها وستون های سیستم باربر جانبی متصل است،ورق جان نامیده می شود.***
- ***ستون ها در سیستم دیوار برشی به اعضای مرزی قائم VBE وتیرها به عنوان اعضای مرزی افقی HBE موسوم هستند.***

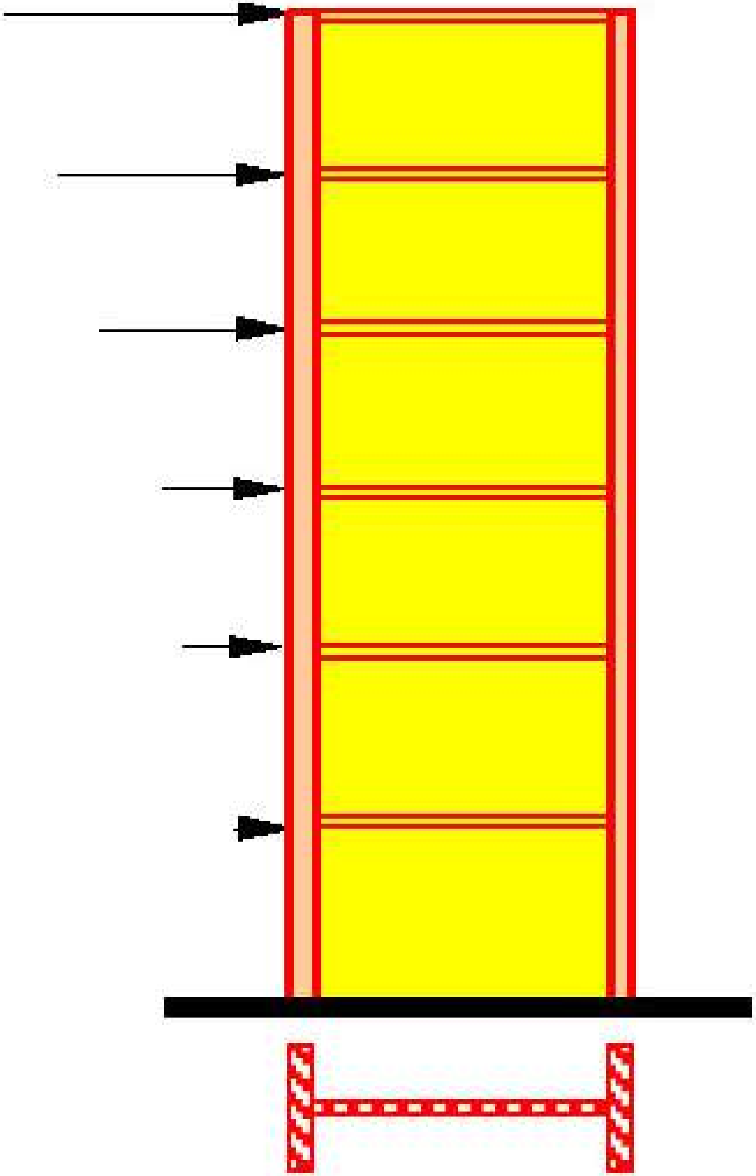
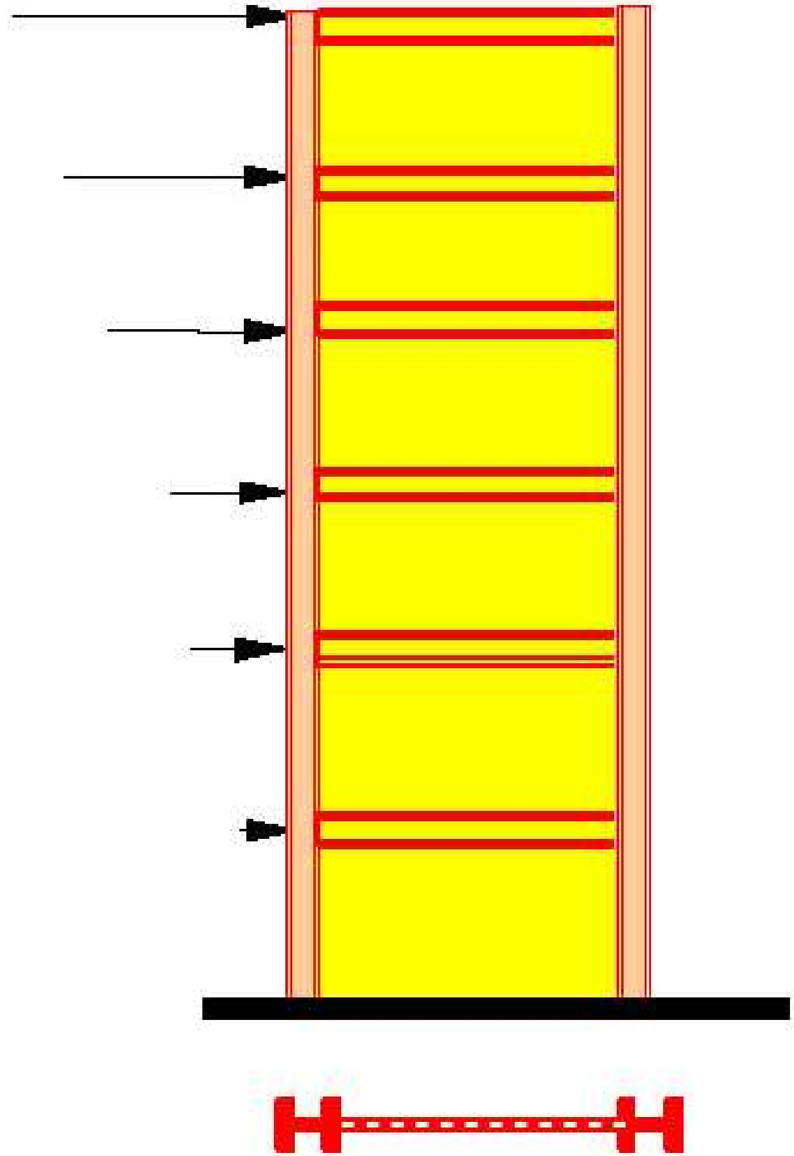
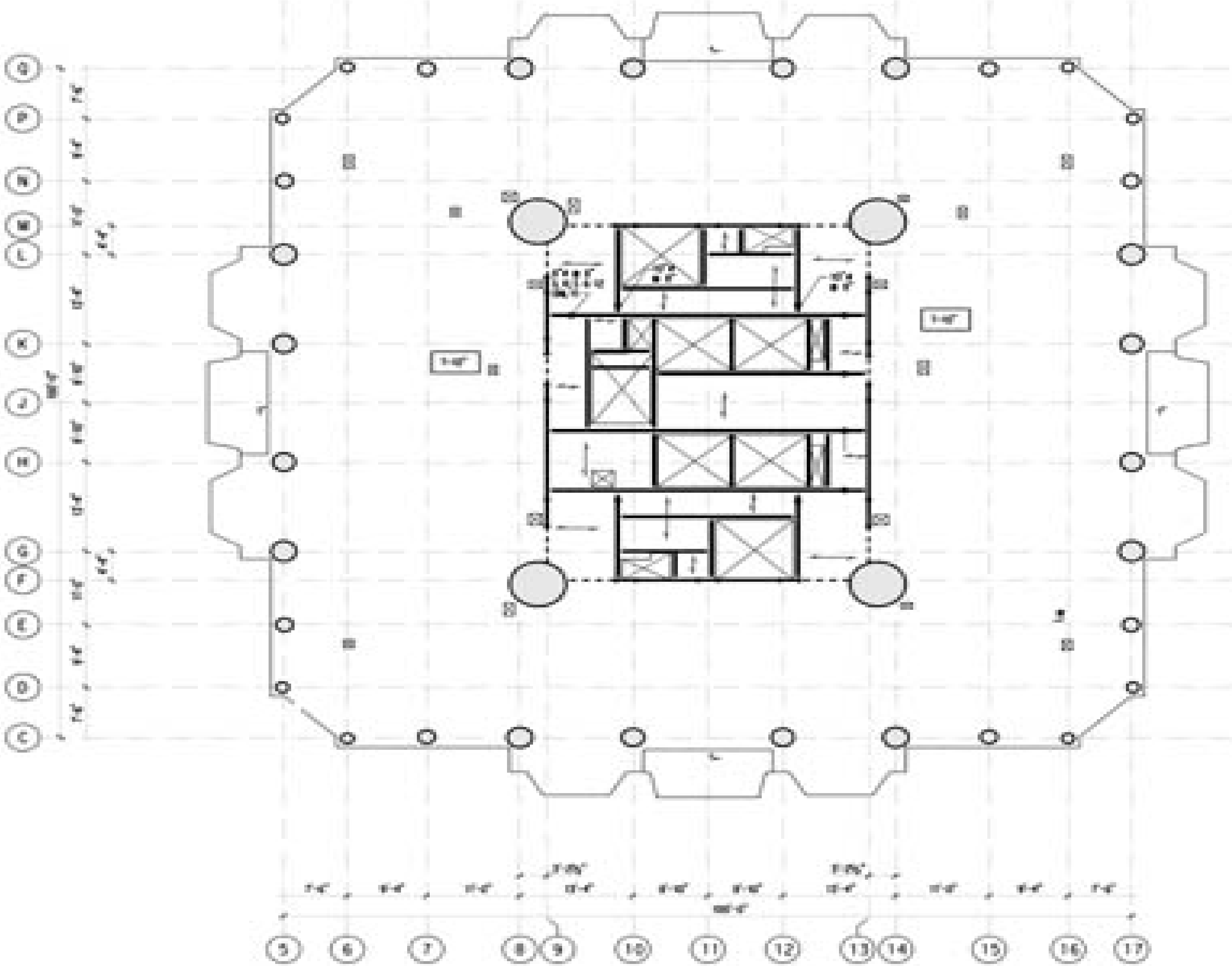


Plate Girder



Steel Plate Shear Wall

The Century, San Francisco –



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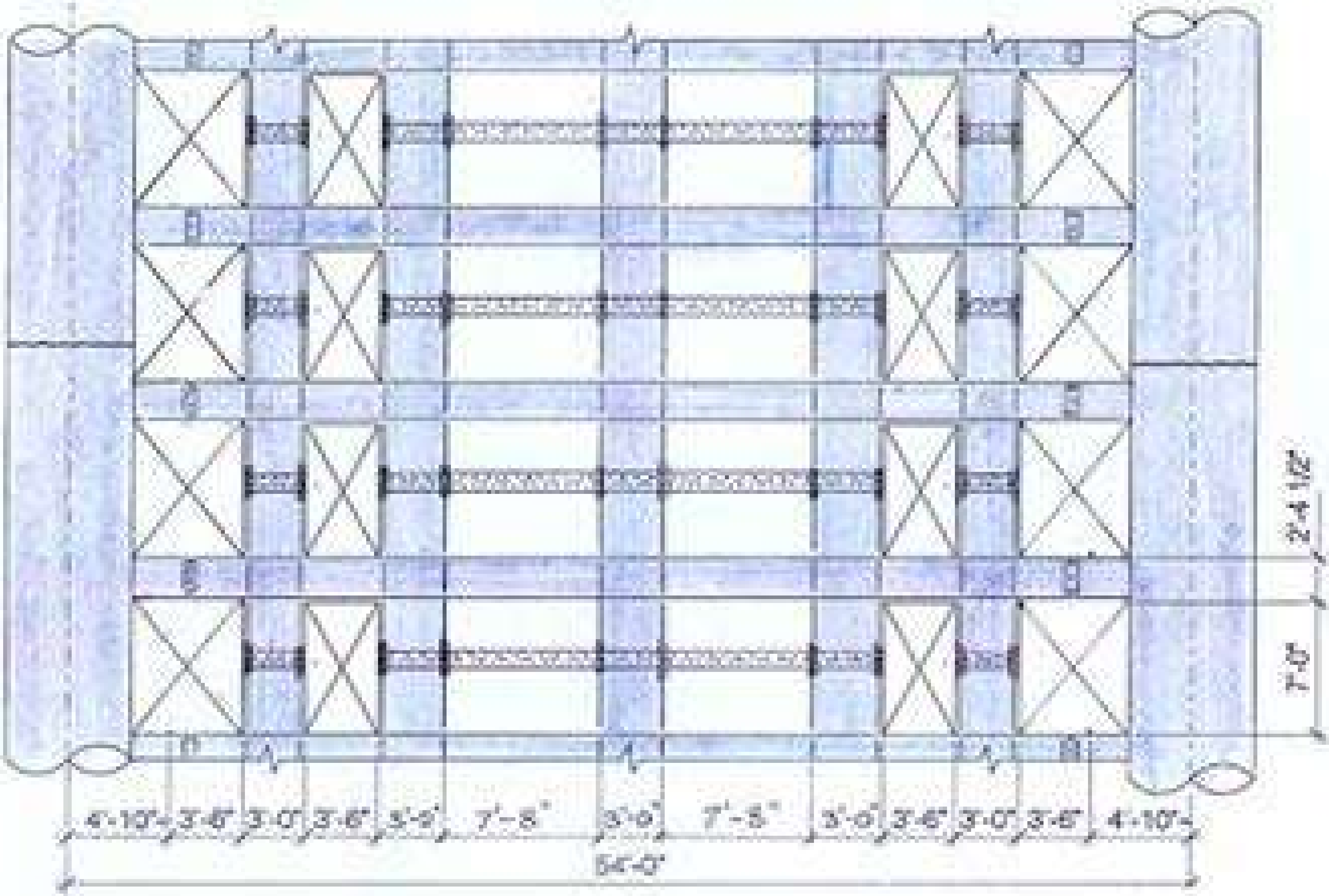
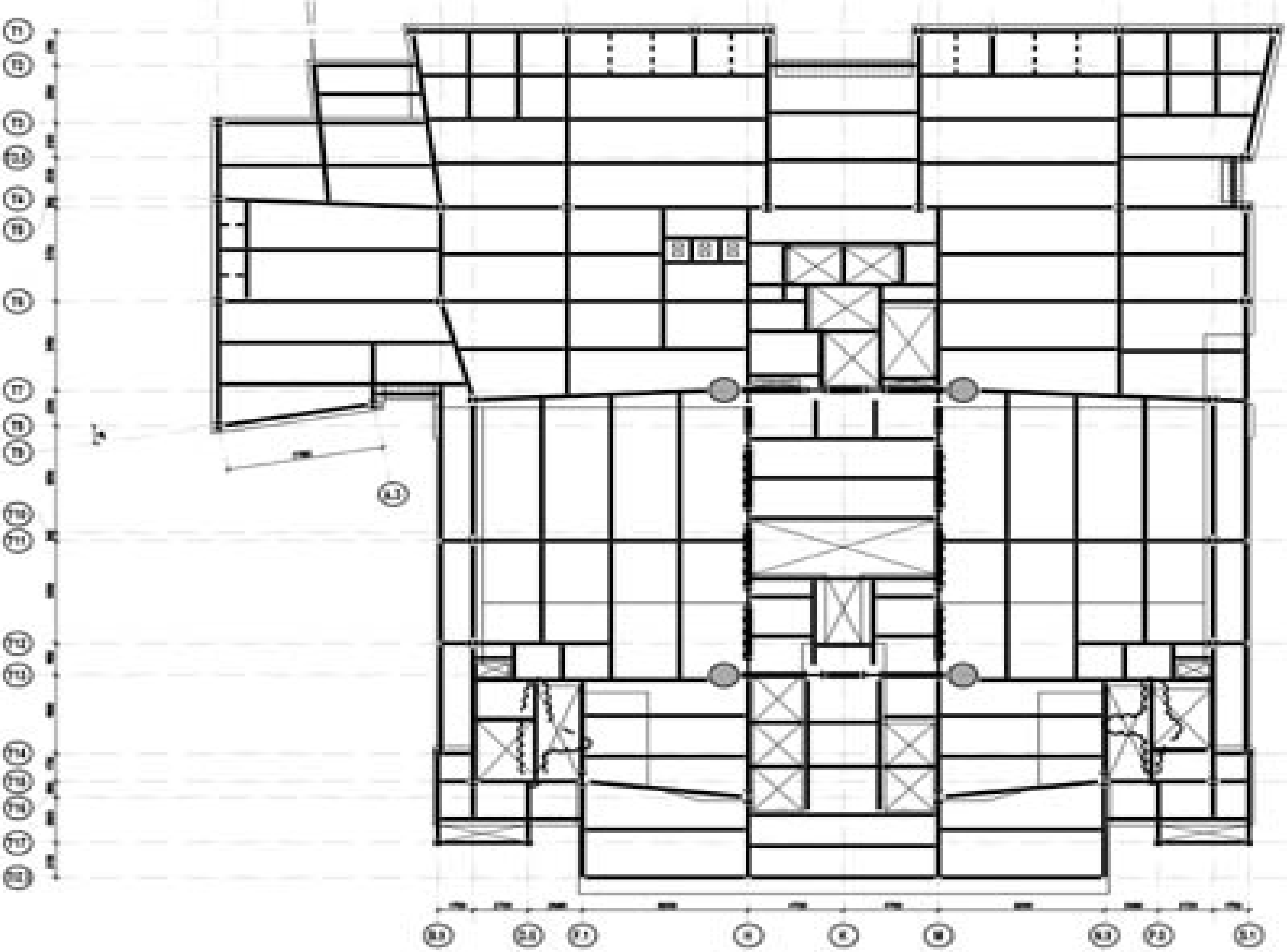
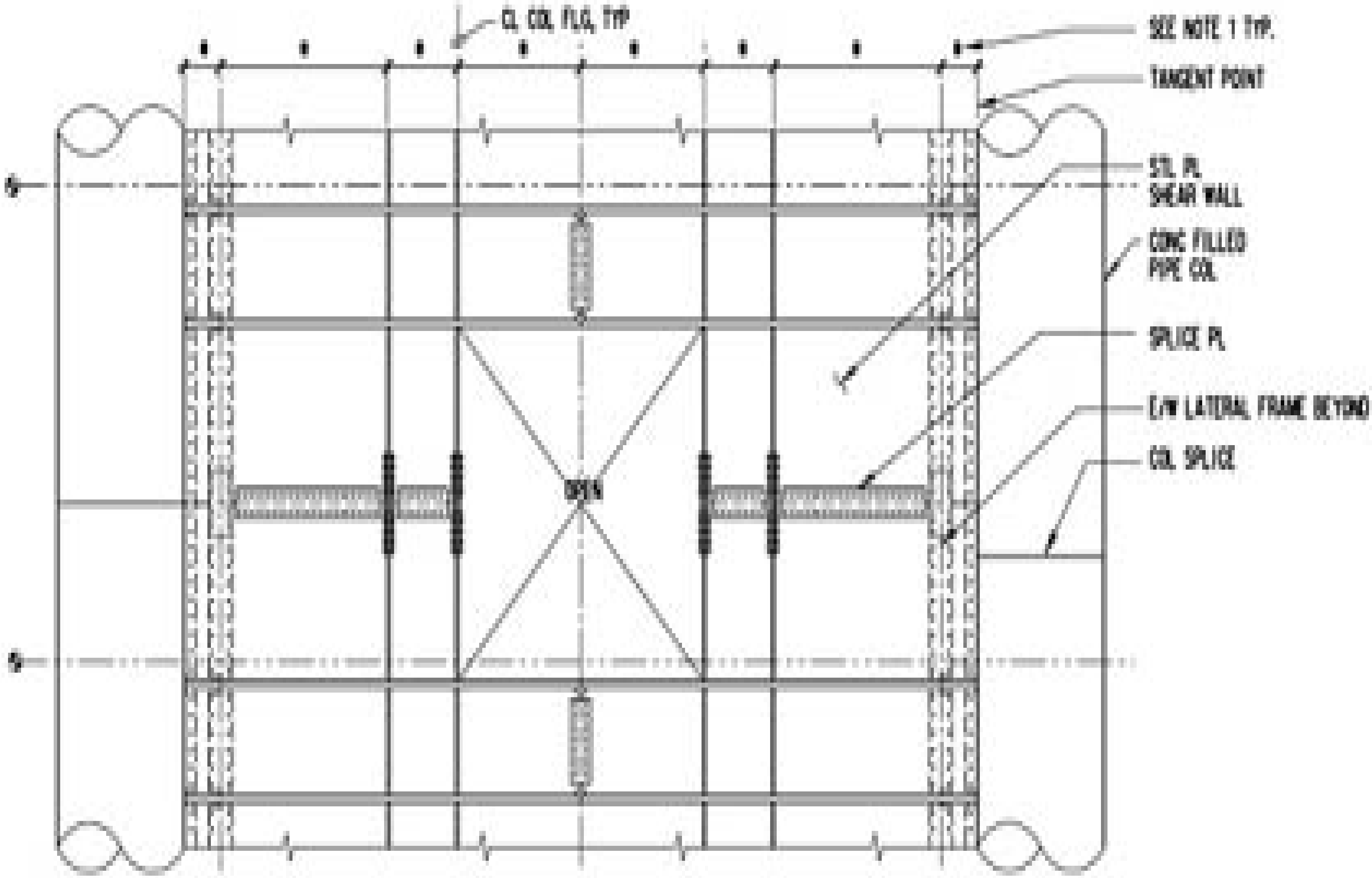


EAST & WEST ELEVATION

U.S. Federal Courthouse, Seattle •





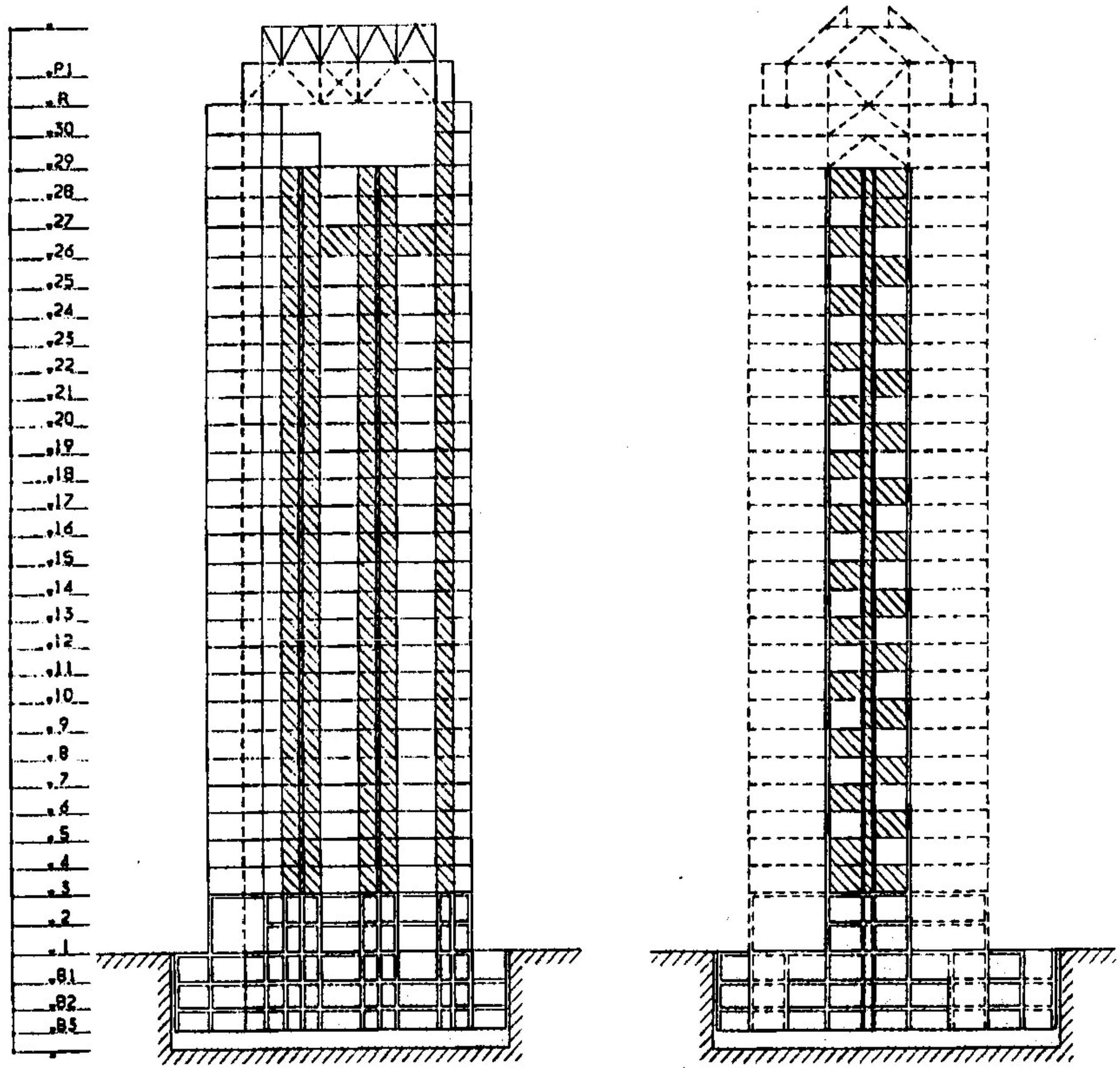
CL COL FLG, TYP
SEE NOTE 1 TYP.
TANGENT POINT
STL PL
SHEAR WALL
CONC FILLED
PIPE COL
SPLICE PL
E/W LATERAL FRAME BEYOND
COL SPLICE
OPEN

NTS

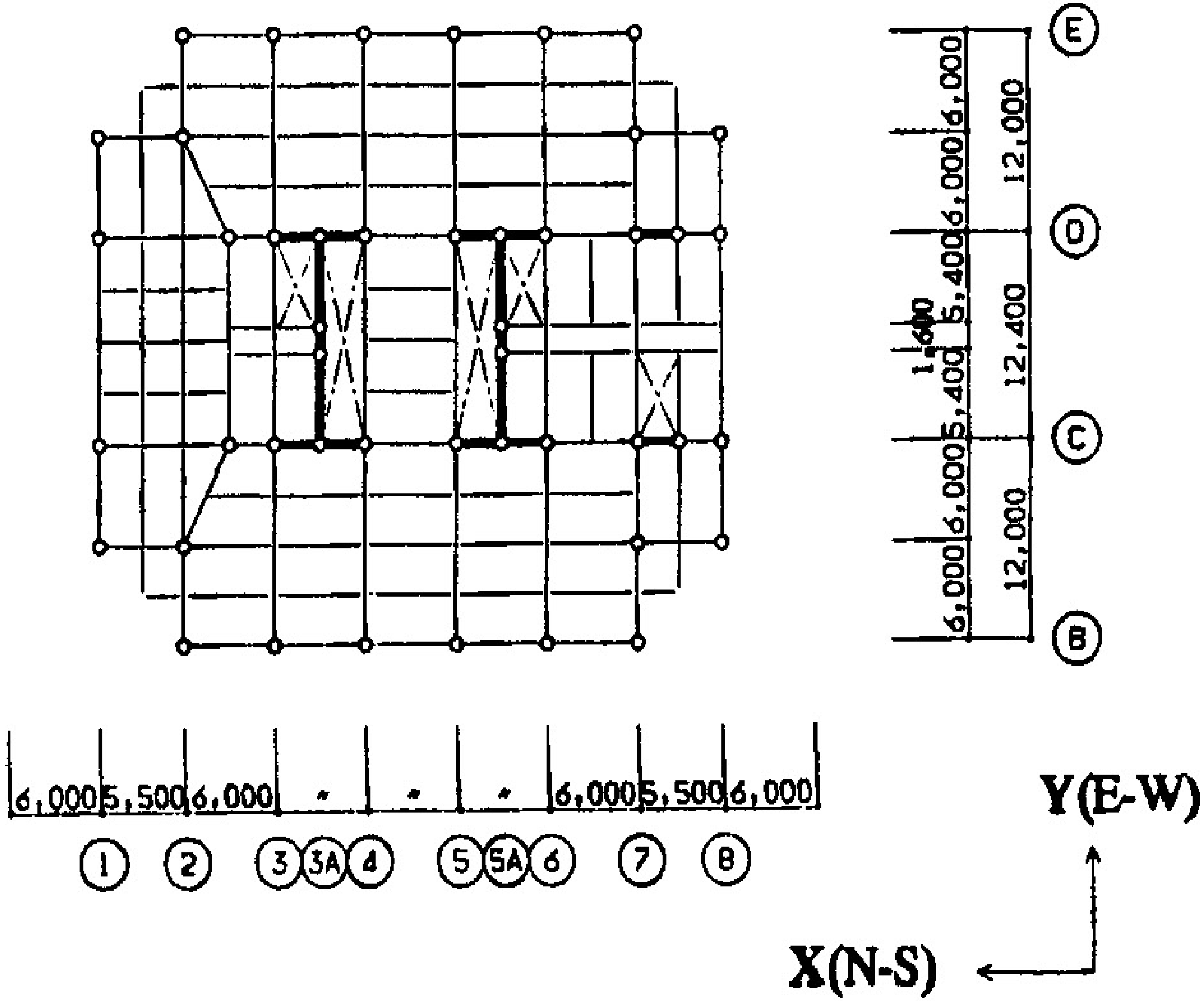
Hyatt Regency Hotel, Dallas •

Kobe Office Building, Kobe •

Kobe City Hall (photo by M. Bruneau).



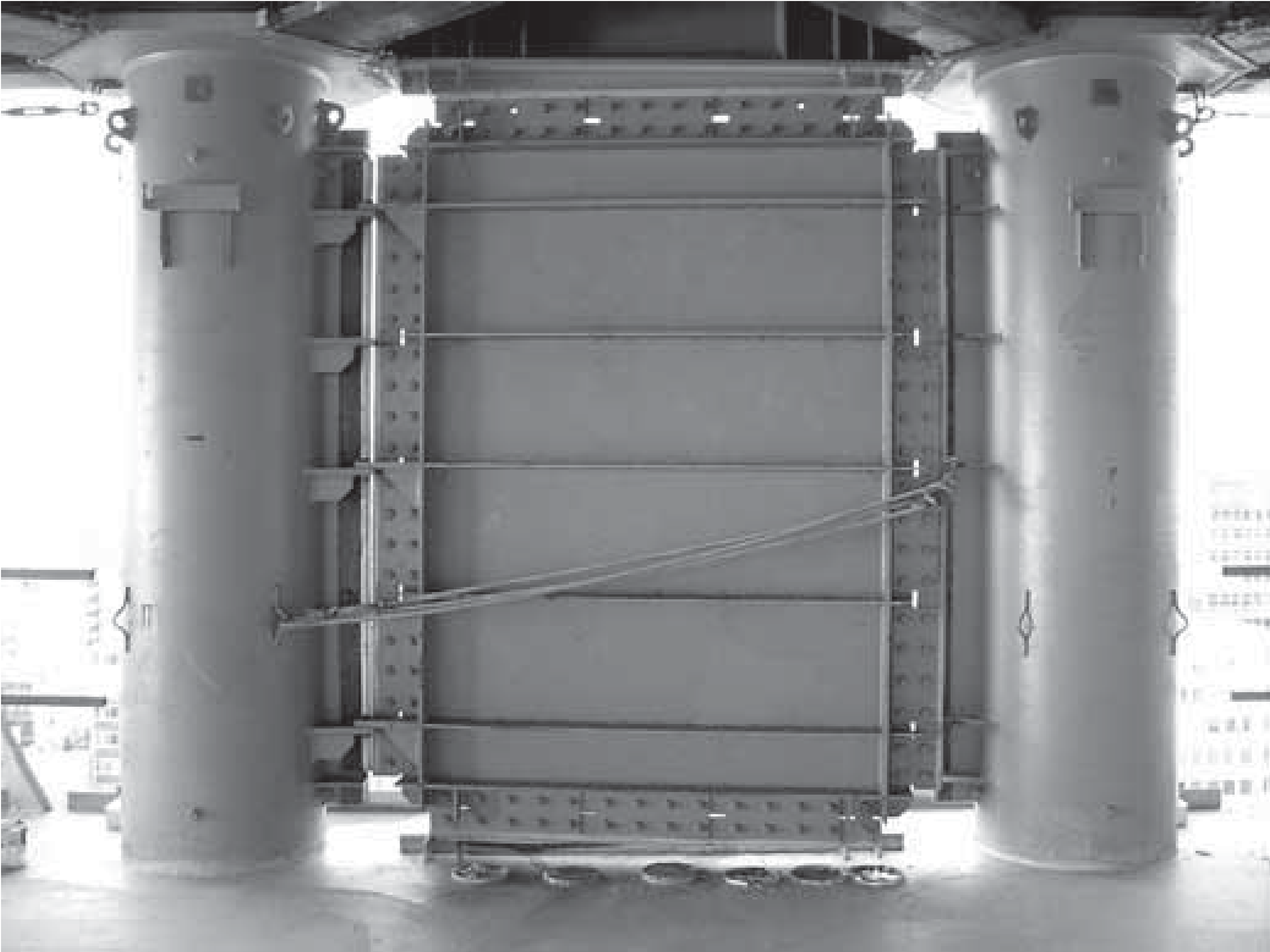
(a) Frame elevation (X(N-S)-direction) (b) Frame elevation (Y(E-W)-direction)

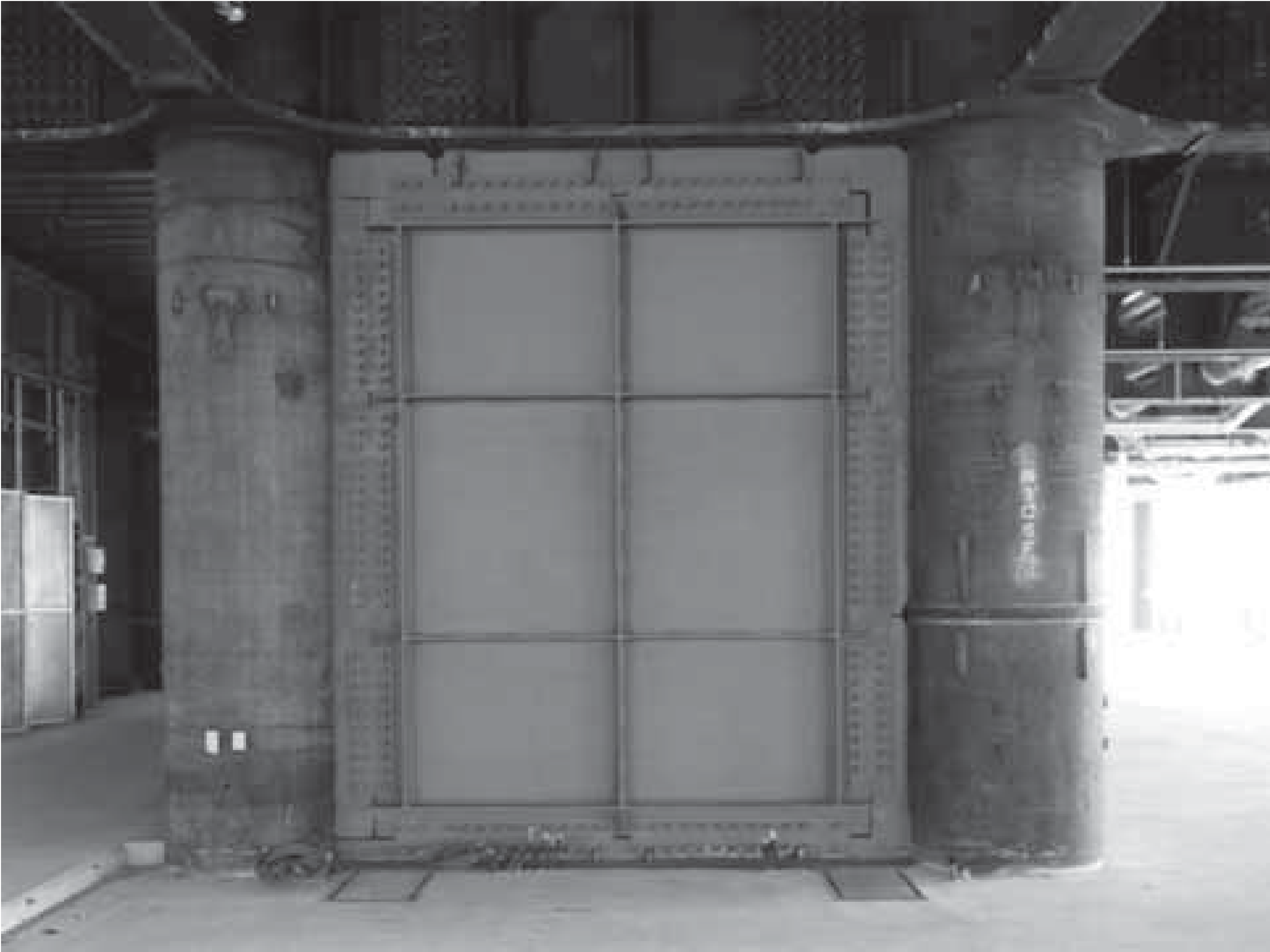


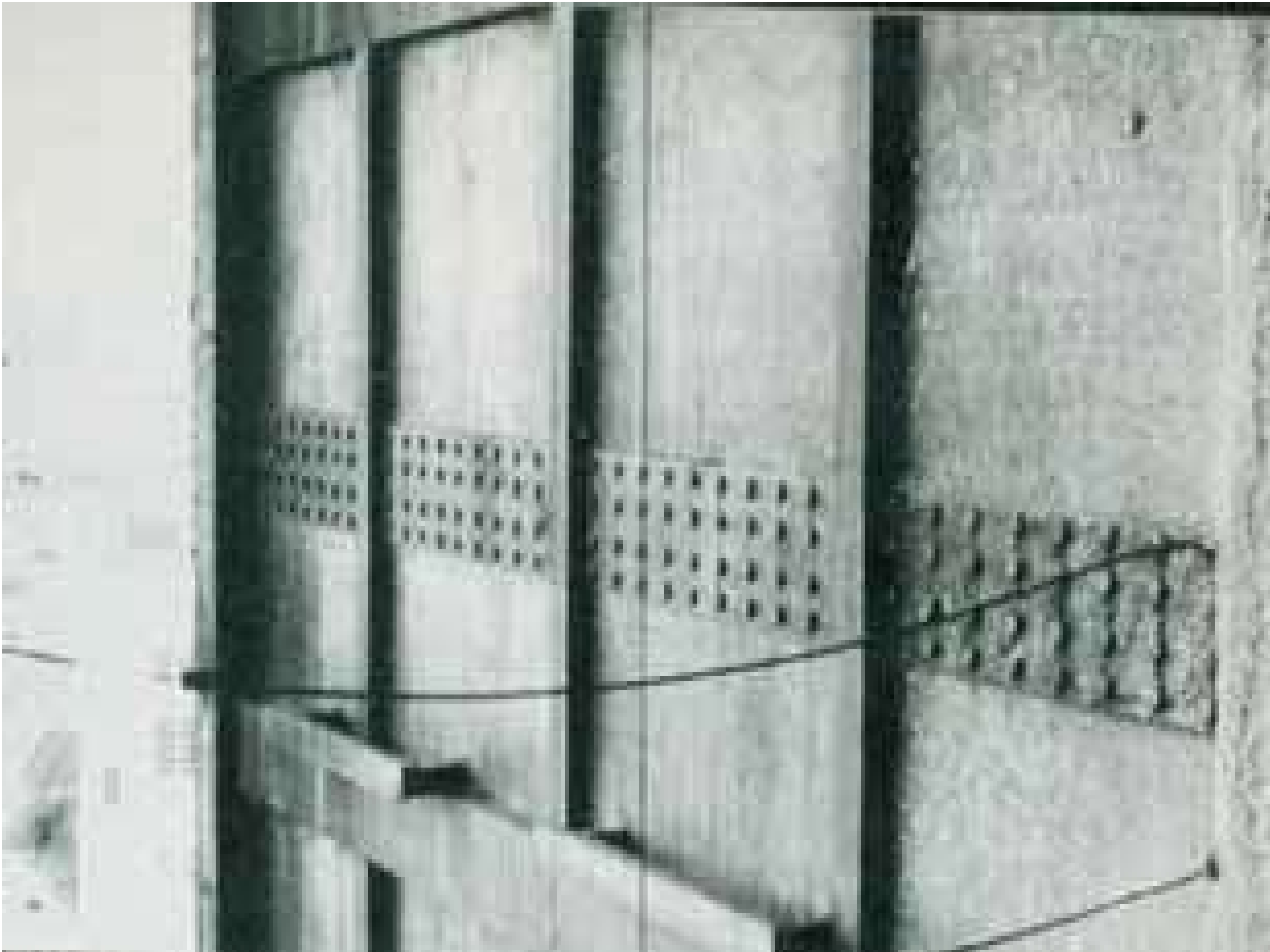
(c) Floor plan

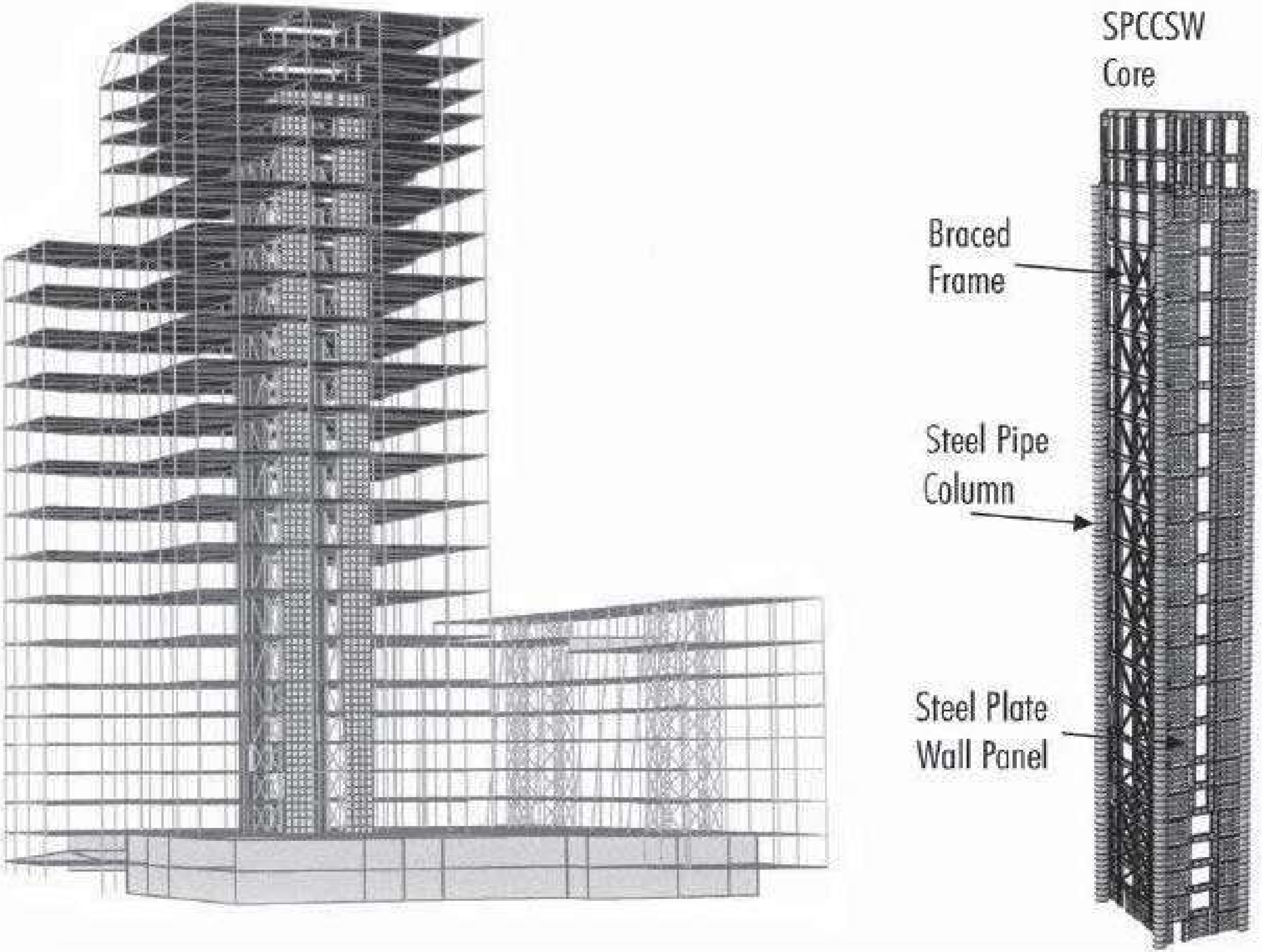
Plan and elevation of Kobe City Hall Building (Fujitana et al., 1996).

Shinjuku Nomura Building (top) and Nippon Steel Building (bottom).

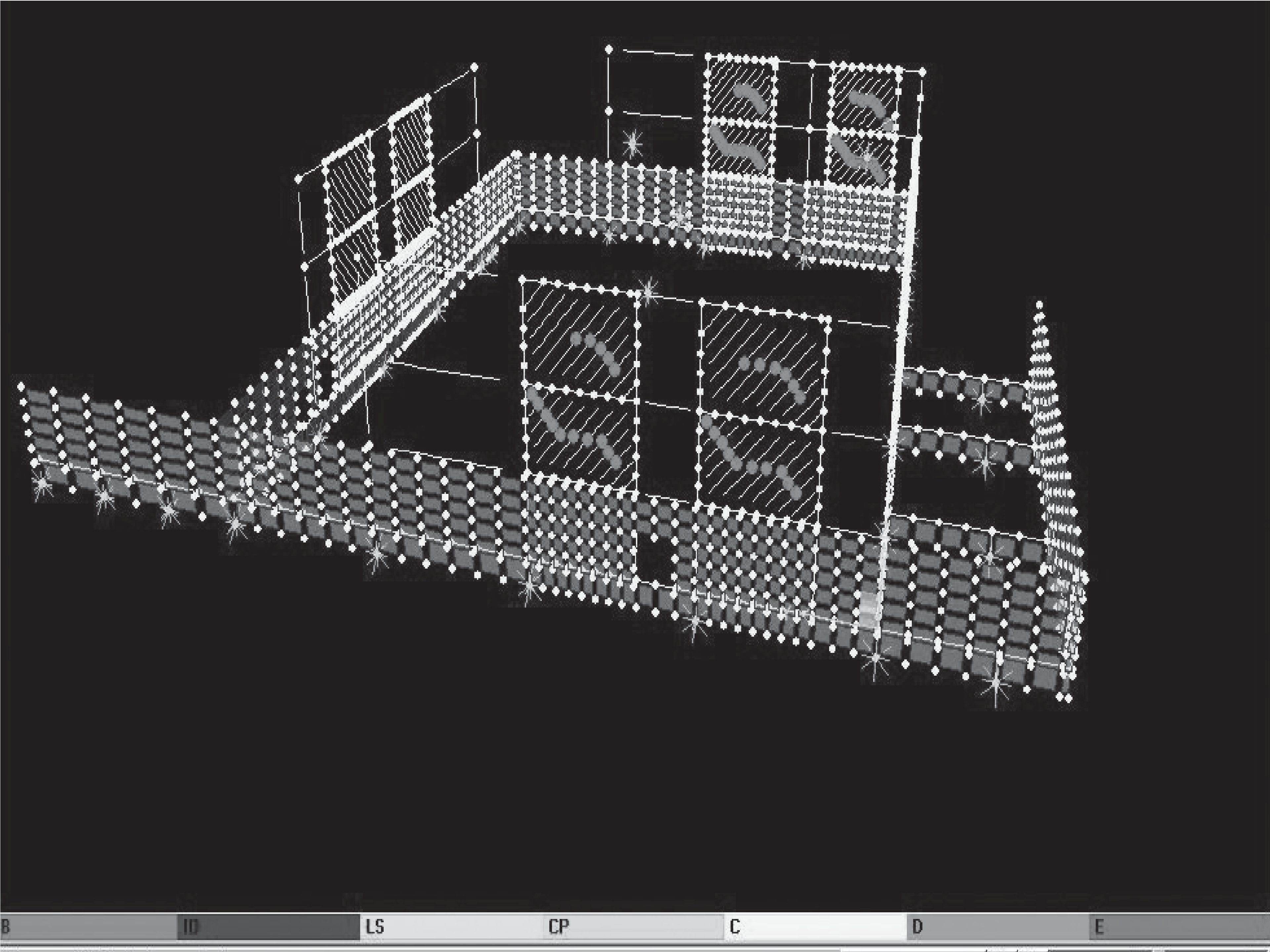








Structural system for U.S. Federal Courthouse, Seattle
(courtesy of John Hooper, Magnusson Klemencic Associates, Seattle, WA).


B
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LS
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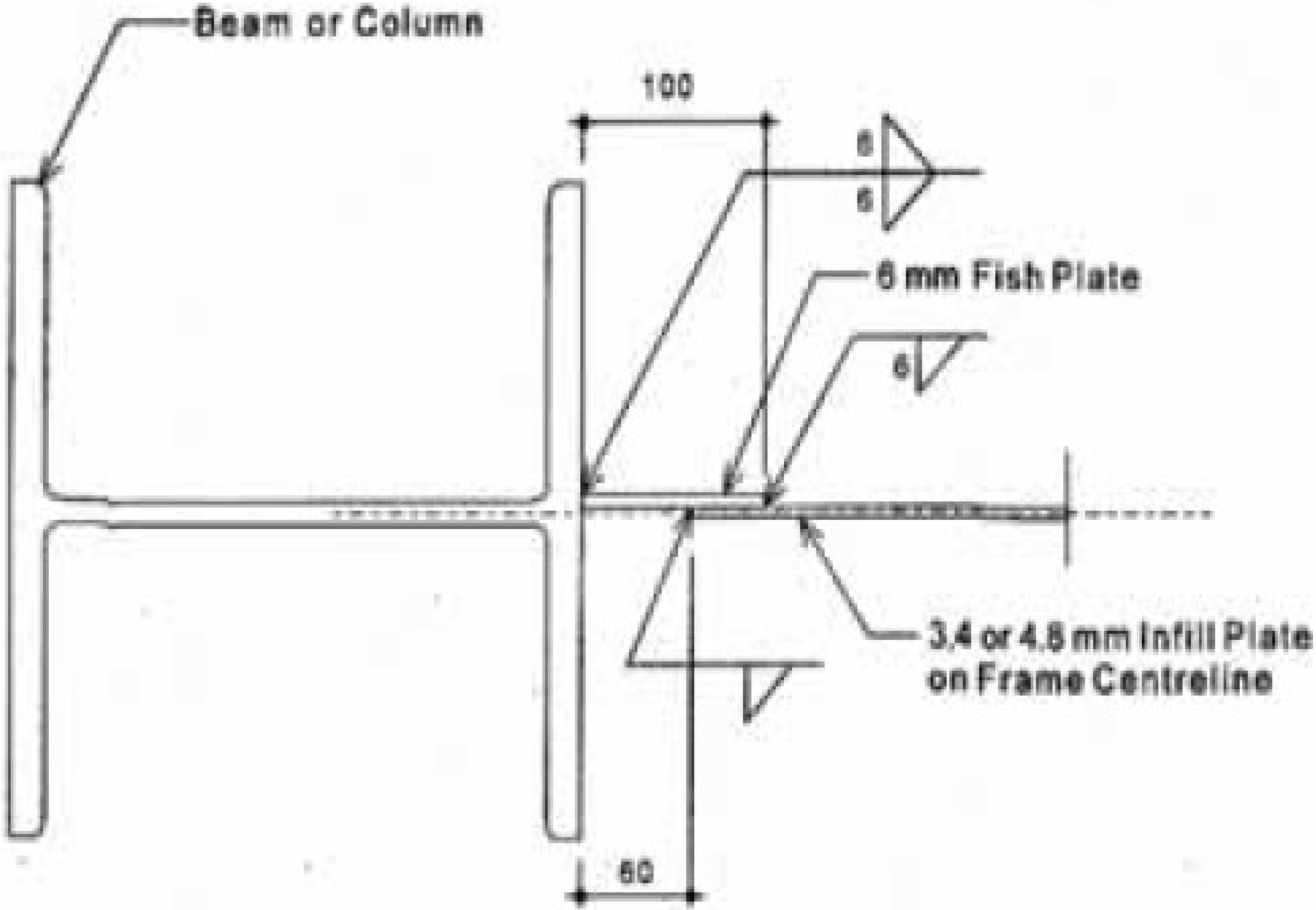
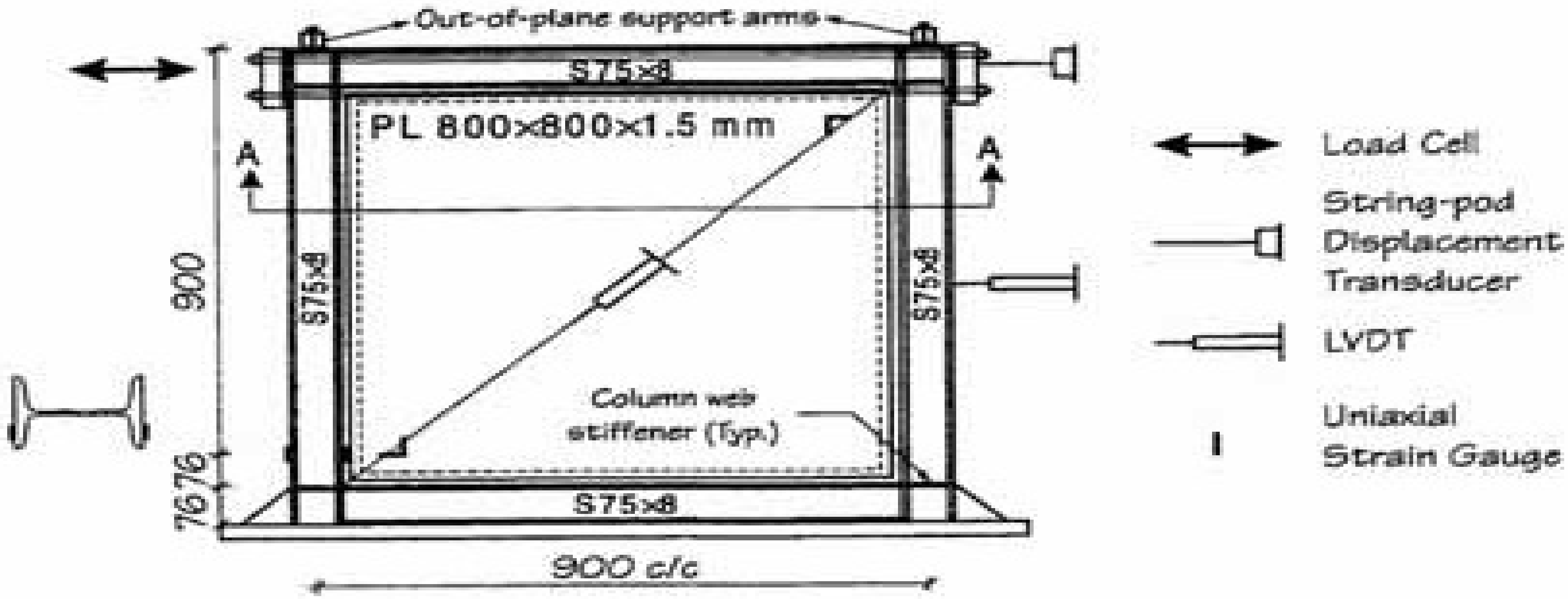
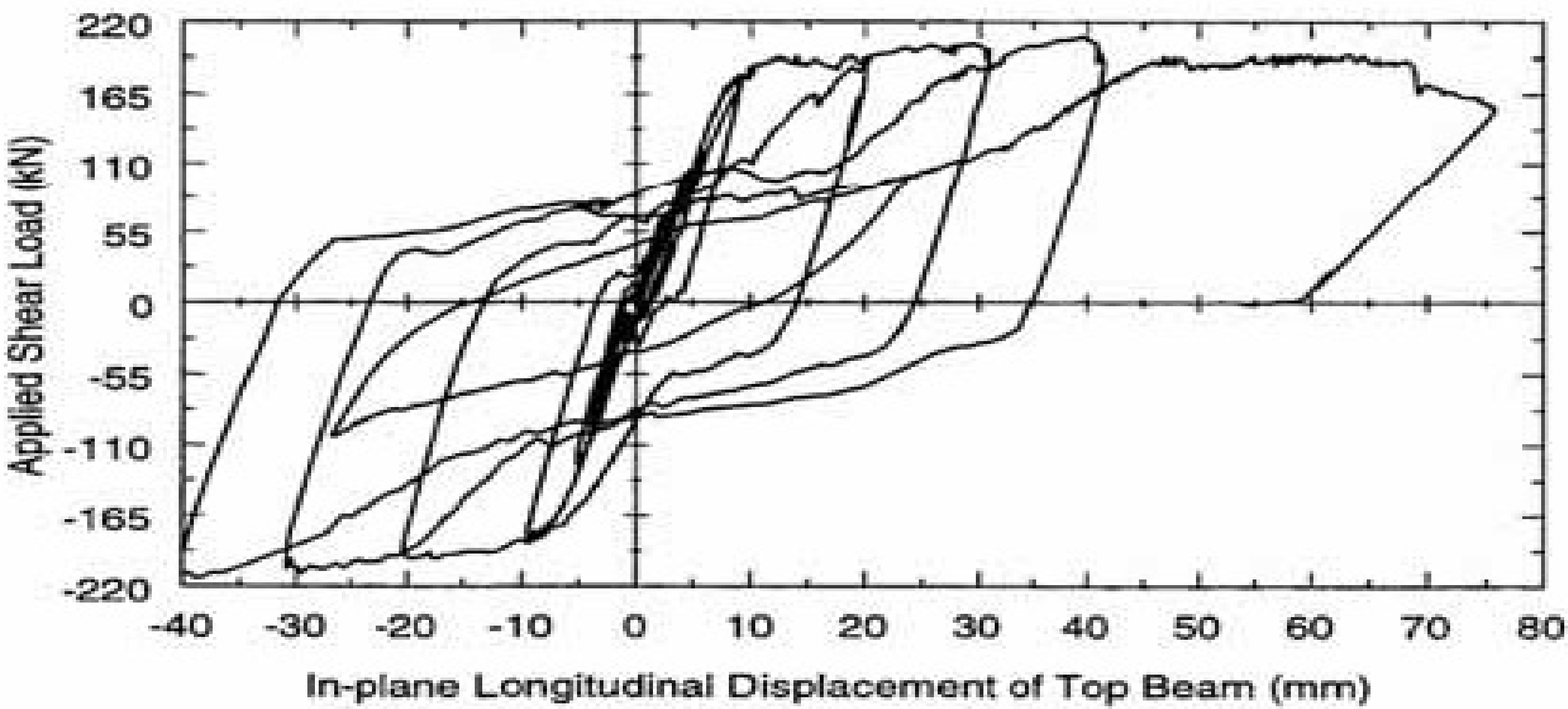
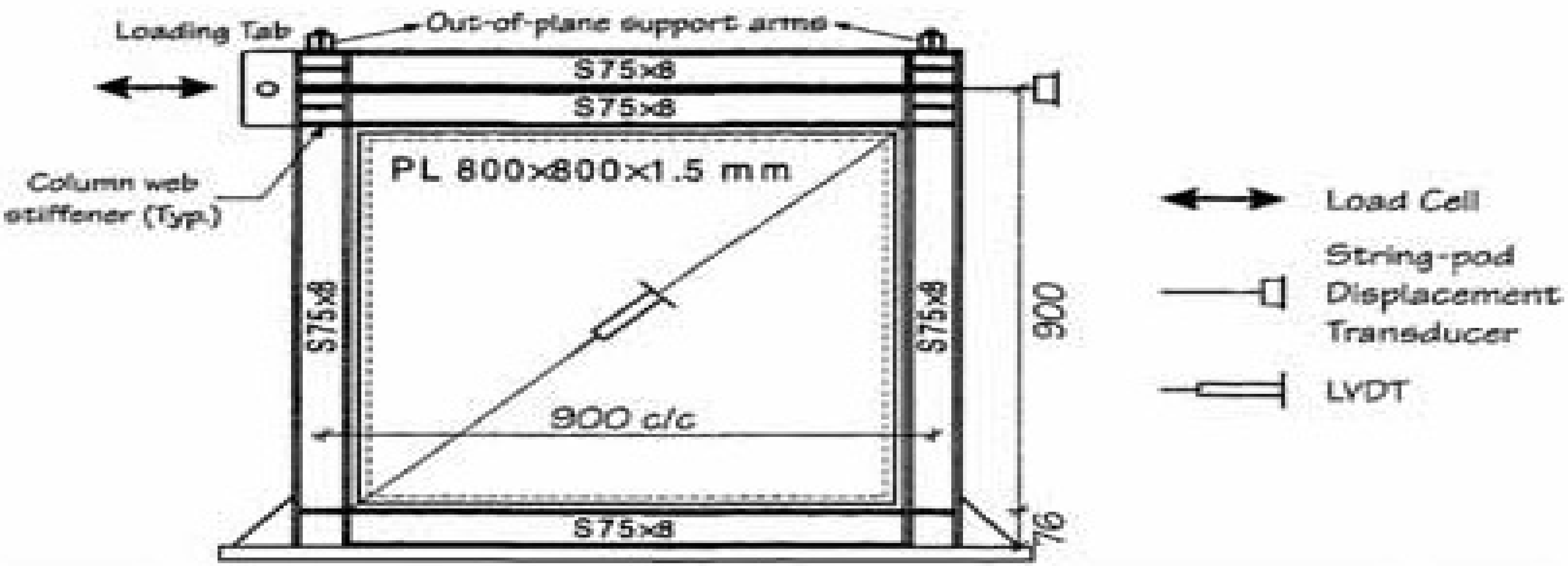
E

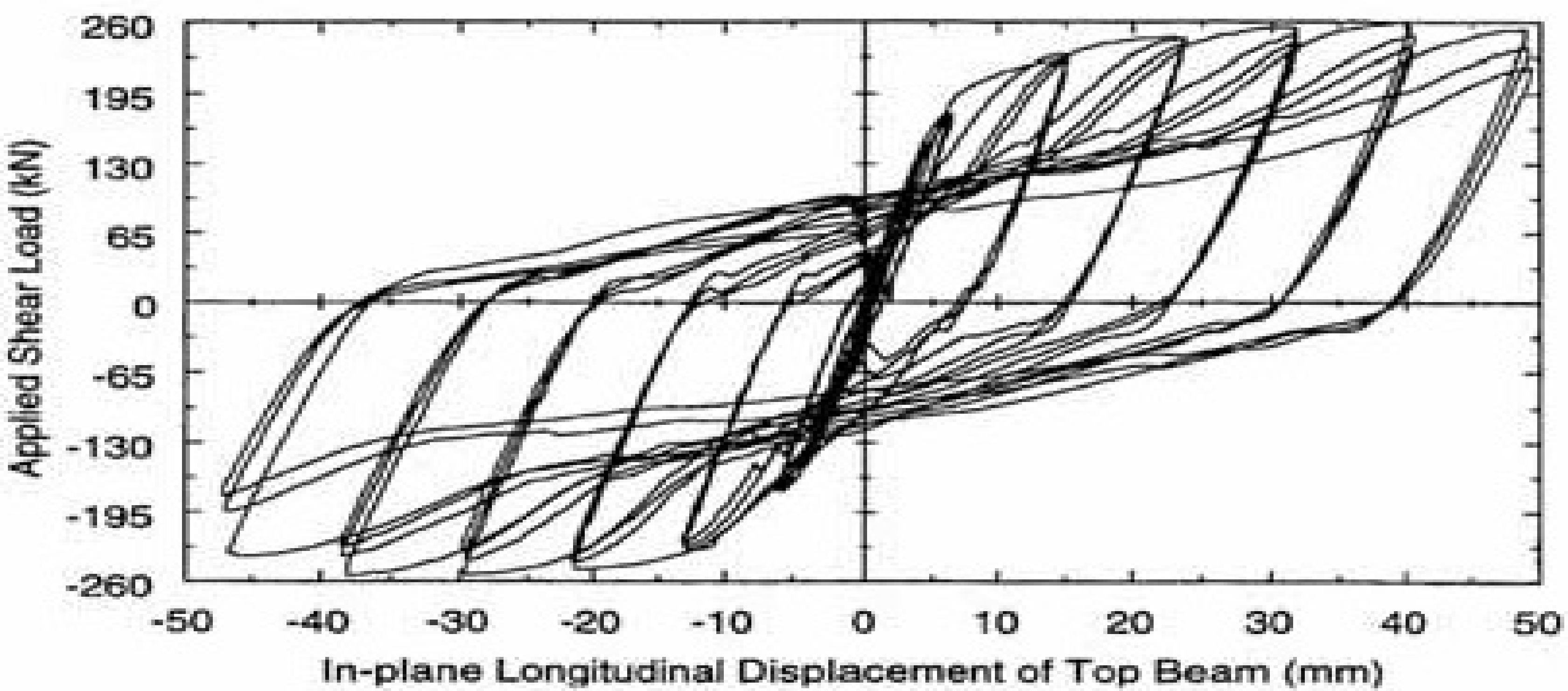

Beam or Column
100
6
6
6 mm Fish Plate
6
3.4 or 4.8 mm Infill Plate
on Frame Centreline
60


Out-of-plane support arms
S75x8
PL 800x800x1.5 mm
A
A
900
S75x8
S75x8
Column web stiffener (Typ.)
76
76
S75x8
900 c/c
Load Cell
String-pod Displacement Transducer
LVDT
Uniaxial Strain Gauge


Applied Shear Load (kN)
220
165
110
55
0
-55
-110
-165
-220
-40
-30
-20
-10
0
10
20
30
40
50
60
70
80
In-plane Longitudinal Displacement of Top Beam (mm)

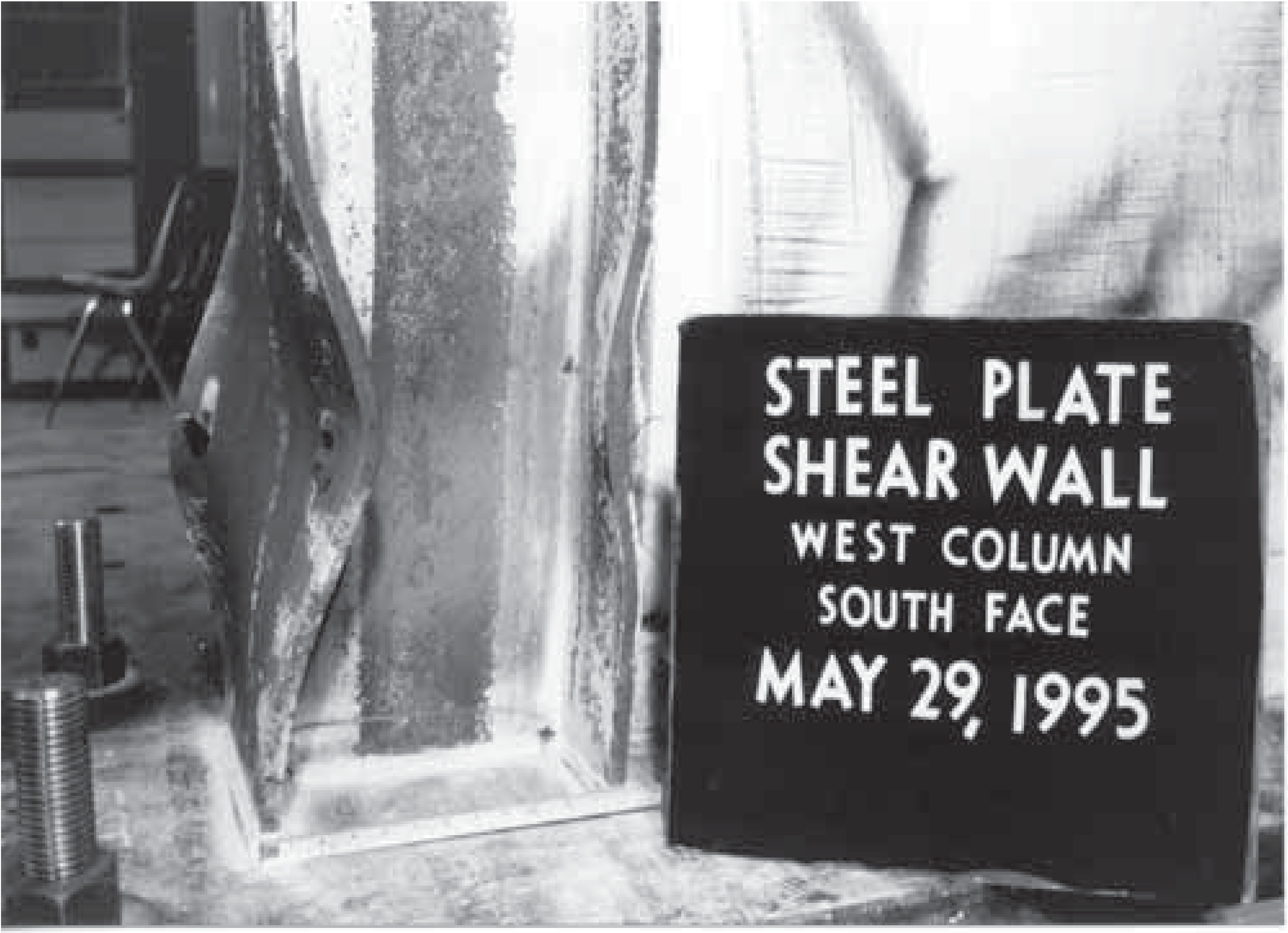


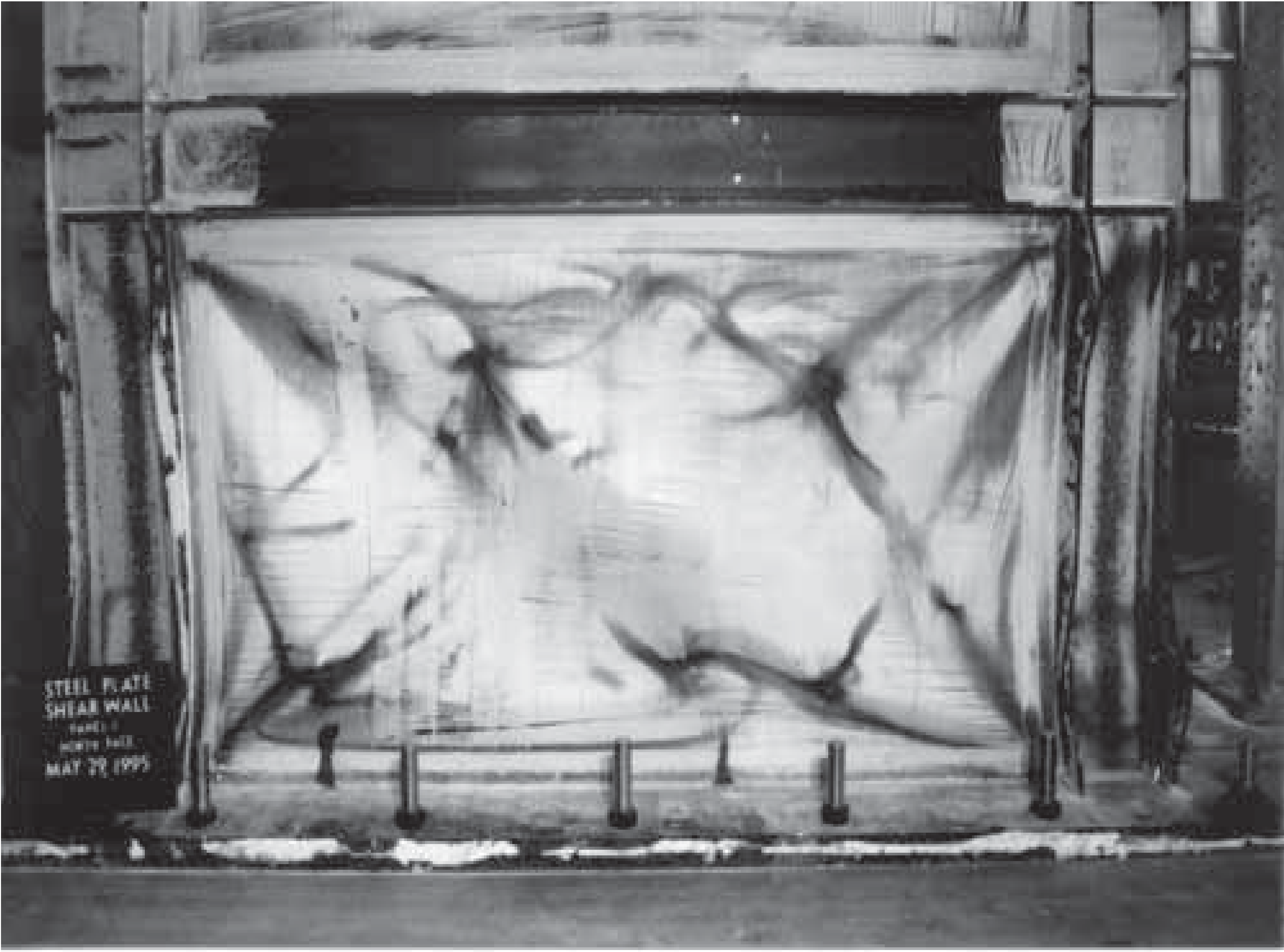
Loading Tab
Out-of-plane support arms
S75x8
S75x8
S75x8
S75x8
S75x8
PL 800x800x1.5 mm
Column web stiffener (Typ.)
900 c/c
900
76
Load Cell
String-pod Displacement Transducer
LVDT



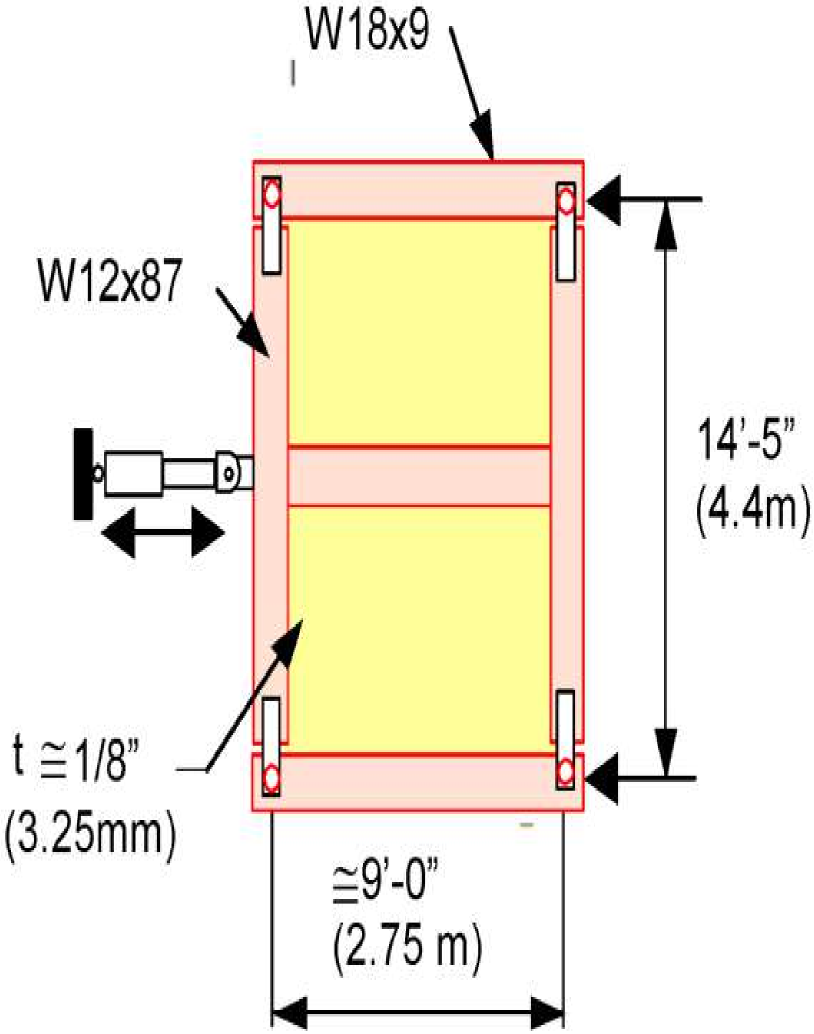
Applied Shear Load (kN)
260
195
130
65
0
-65
-130
-195
-260
-50
-40
-30
-20
-10
0
10
20
30
40
50
In-plane Longitudinal Displacement of Top Beam (mm)


PANEL 1
SOUTH FACE
MAY 29, 1995

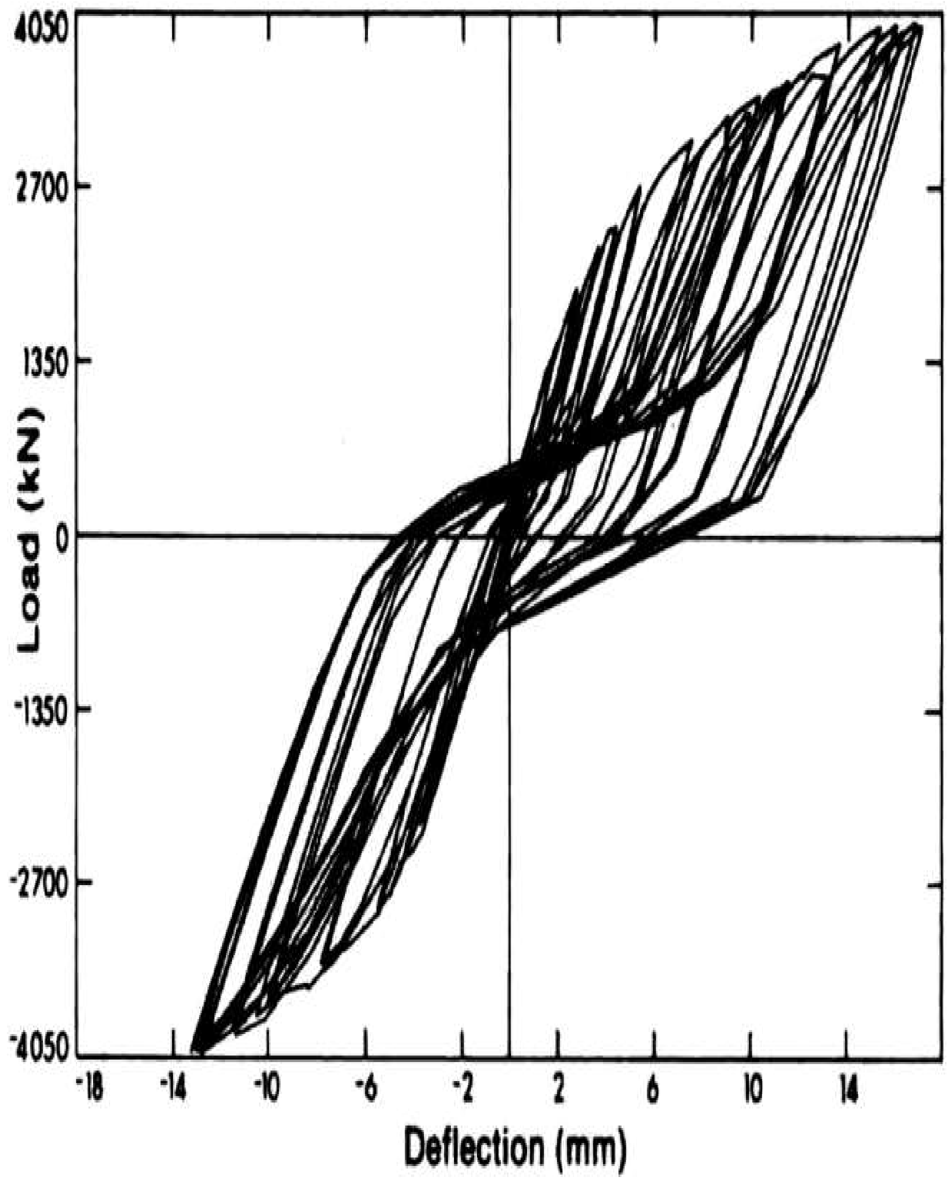

STEEL PLATE
SHEAR WALL
WEST COLUMN
SOUTH FACE
MAY 29, 1995


STEEL PLATE
SHEAR WALL

- نتایج آزمایشات انجام گرفته برروی دیوارهای برشی فولادی تحت بارهای چرخه ای نشانگر سختی زیاد،مقاومت کافی،شکل پذیری مناسب واستهلاک زیاد انرژی حاصل از زلزله در این سیستم باربر جانبی است.
- در نمودار نیرو–تغییر مکان نمونه مورد آزمایش،افزایش مقاومت پس از اولین تسلیم بیش از ۲ ومیزان شکل پذیری ۳/۵ مشاهده می شود.

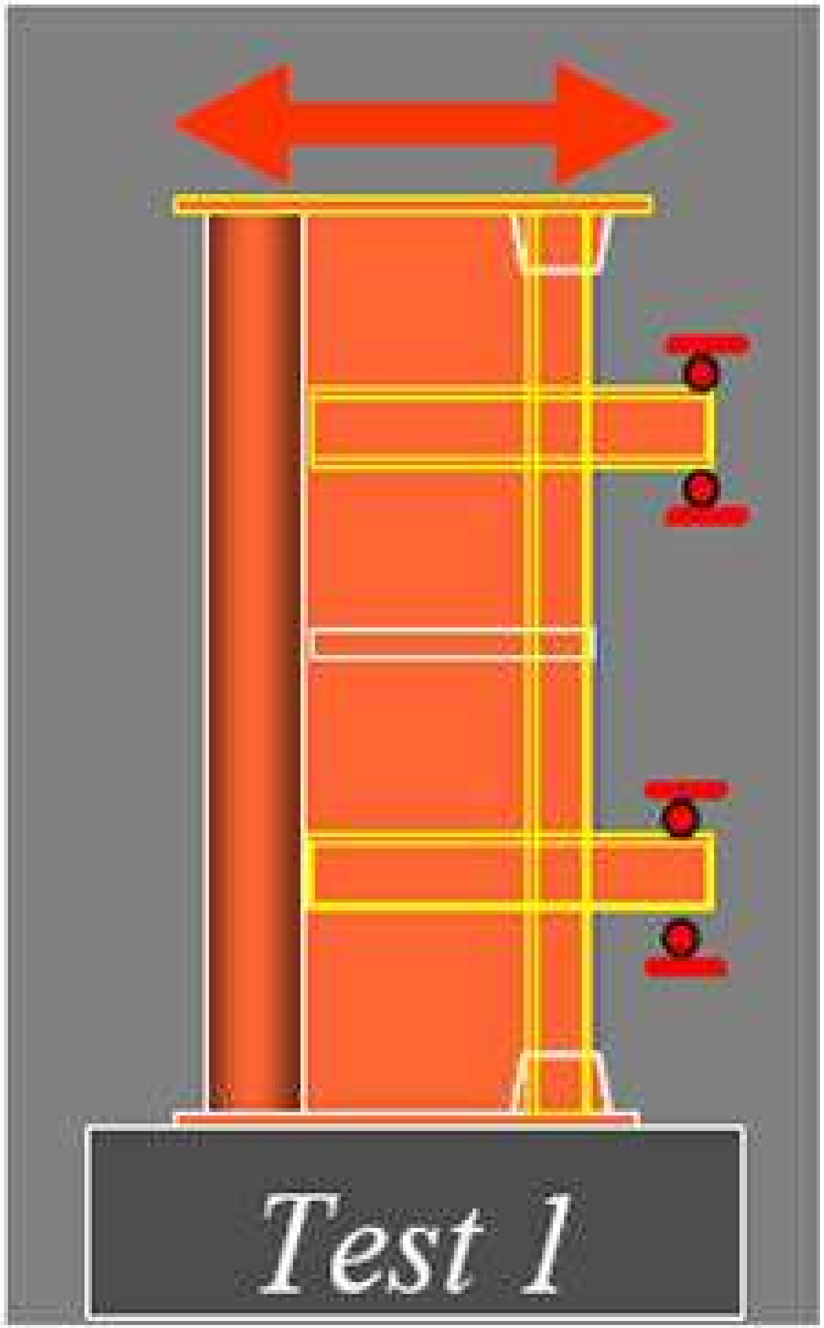


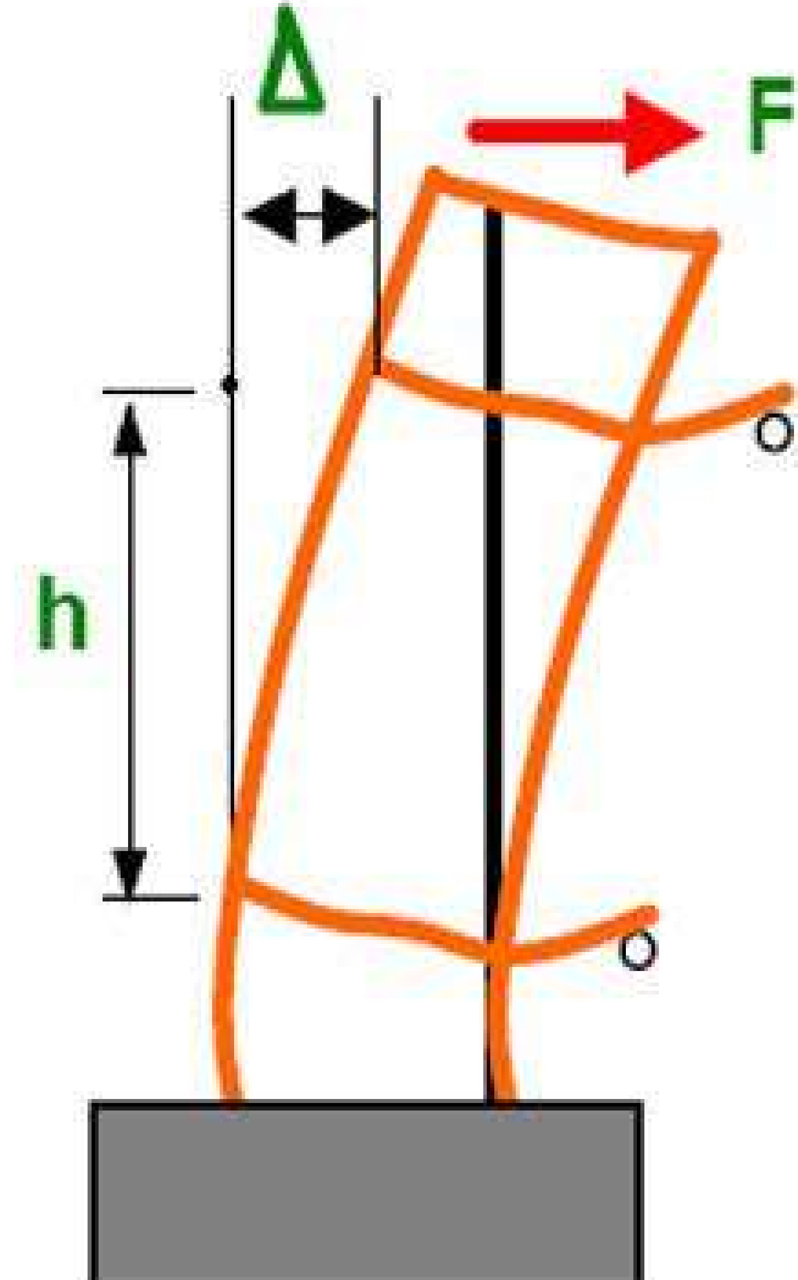
Test Specimen

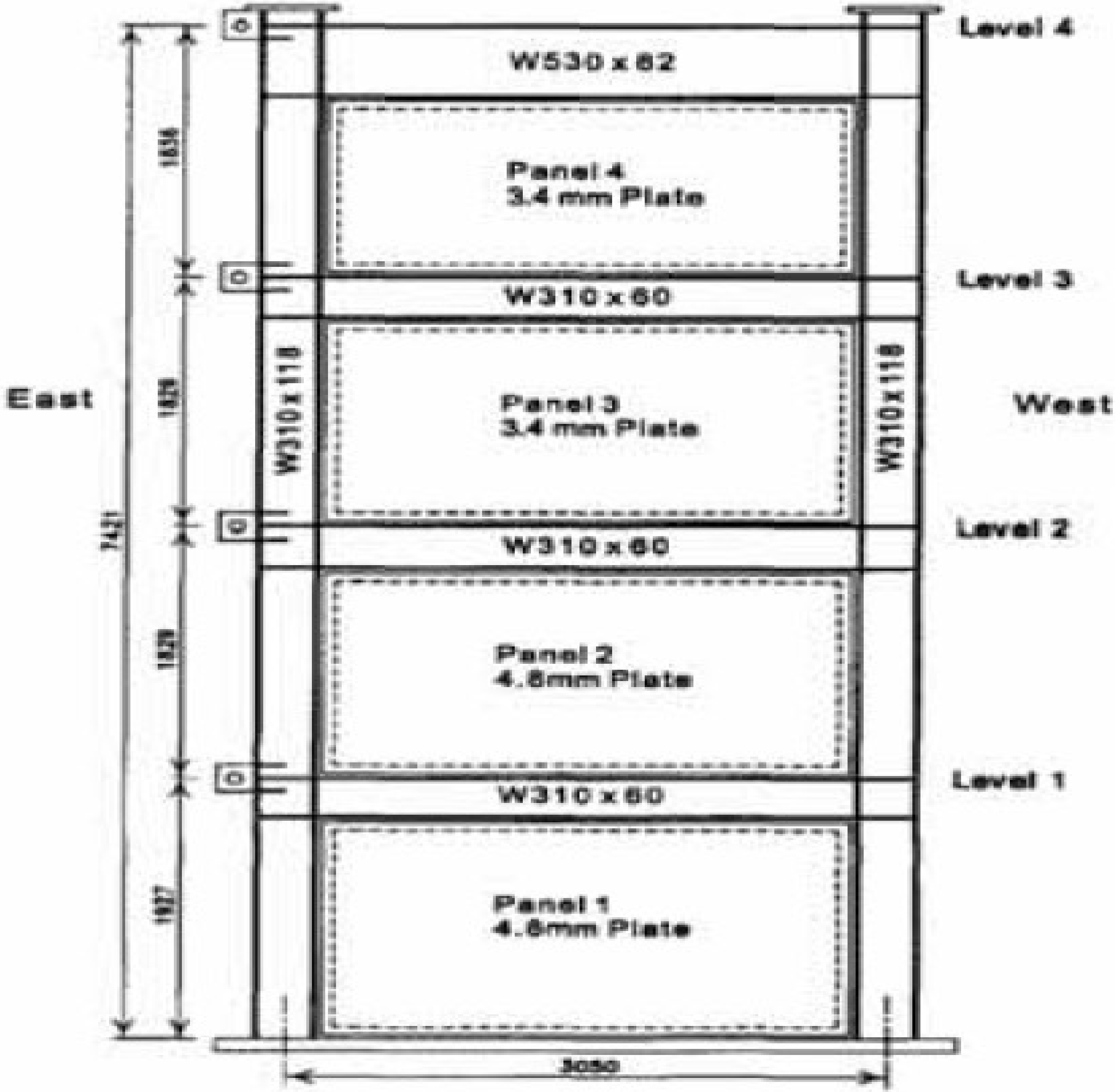


Load vs. Displacement

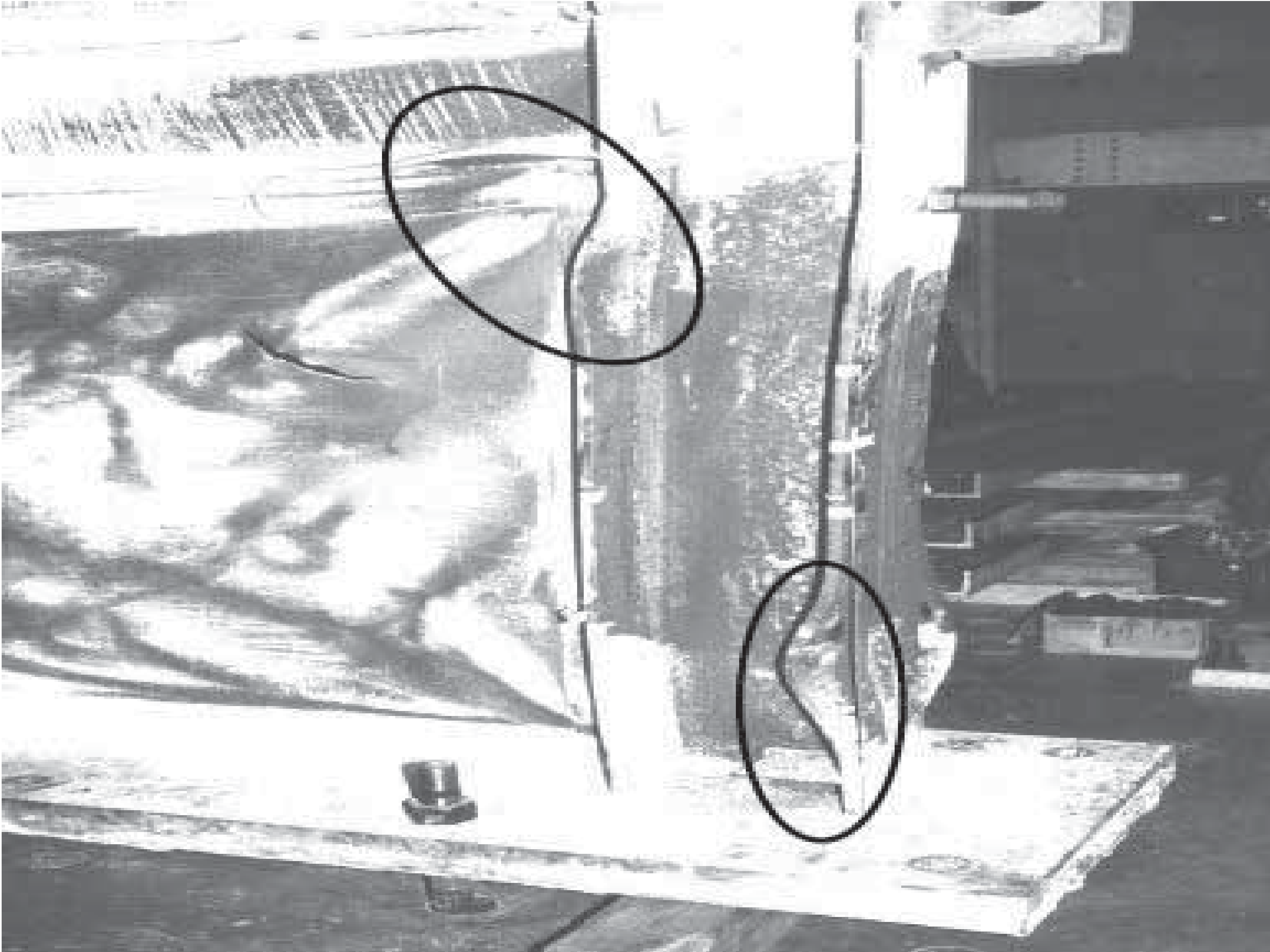
(Curves from: Timler and Kulak, 1983)

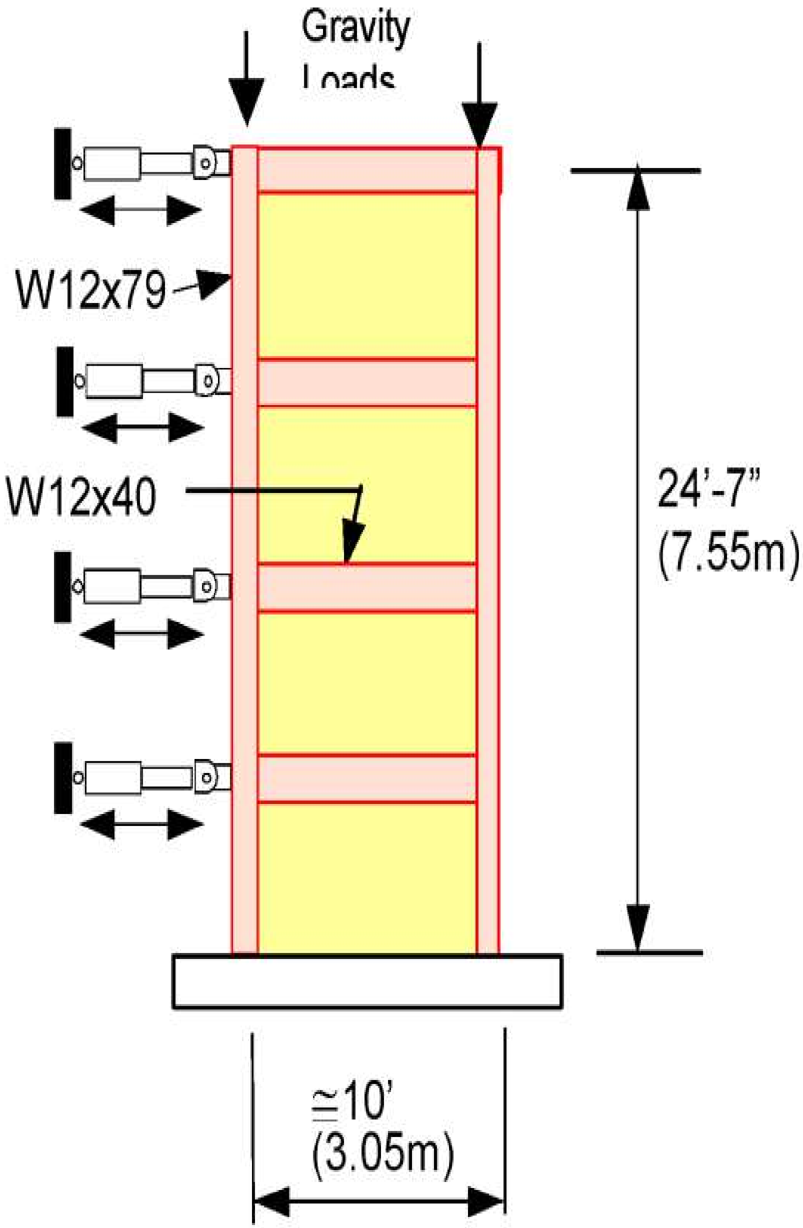

Test 1


Δ
F
h

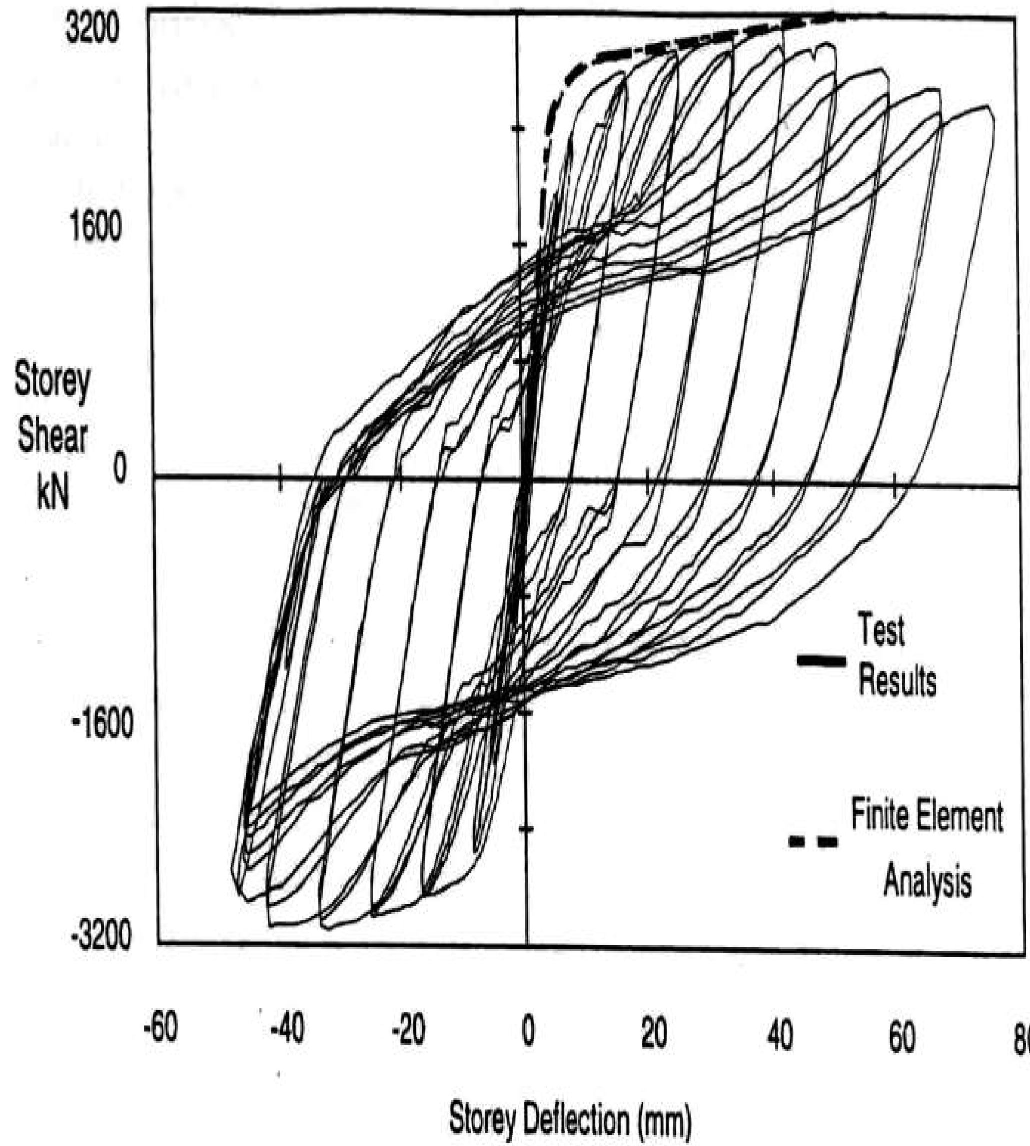

Level 4
W530 x 82
Panel 4
3.4 mm Plate
Level 3
W310 x 60
East
W310x118
Panel 3
3.4 mm Plate
W310x118
West
Level 2
W310 x 60
Panel 2
4.8mm Plate
Level 1
W310 x 60
Panel 1
4.8mm Plate
3050

- نتایج بارگذاری چرخه ای برروی یک دیوار برشی ۴ طبقه نشان داده است که خرابی در محل اتصال ستون به کف ستون که تحت اثر گرمای ناشی از جوشکاری قرار گرفته بود بوجود آمده است.
- علت خرابی مربوط به کمانش موضعی ستون در اثر دامنه تغییر شکل های زیاد بال ستون در سیکل بیستم بارگذاری چرخه ای بوده است.
- نمونه مورد آزمایش تا قبل از گسیختگی رفتار بسیار نرم وشکل پذیری از خود نشان داده است.



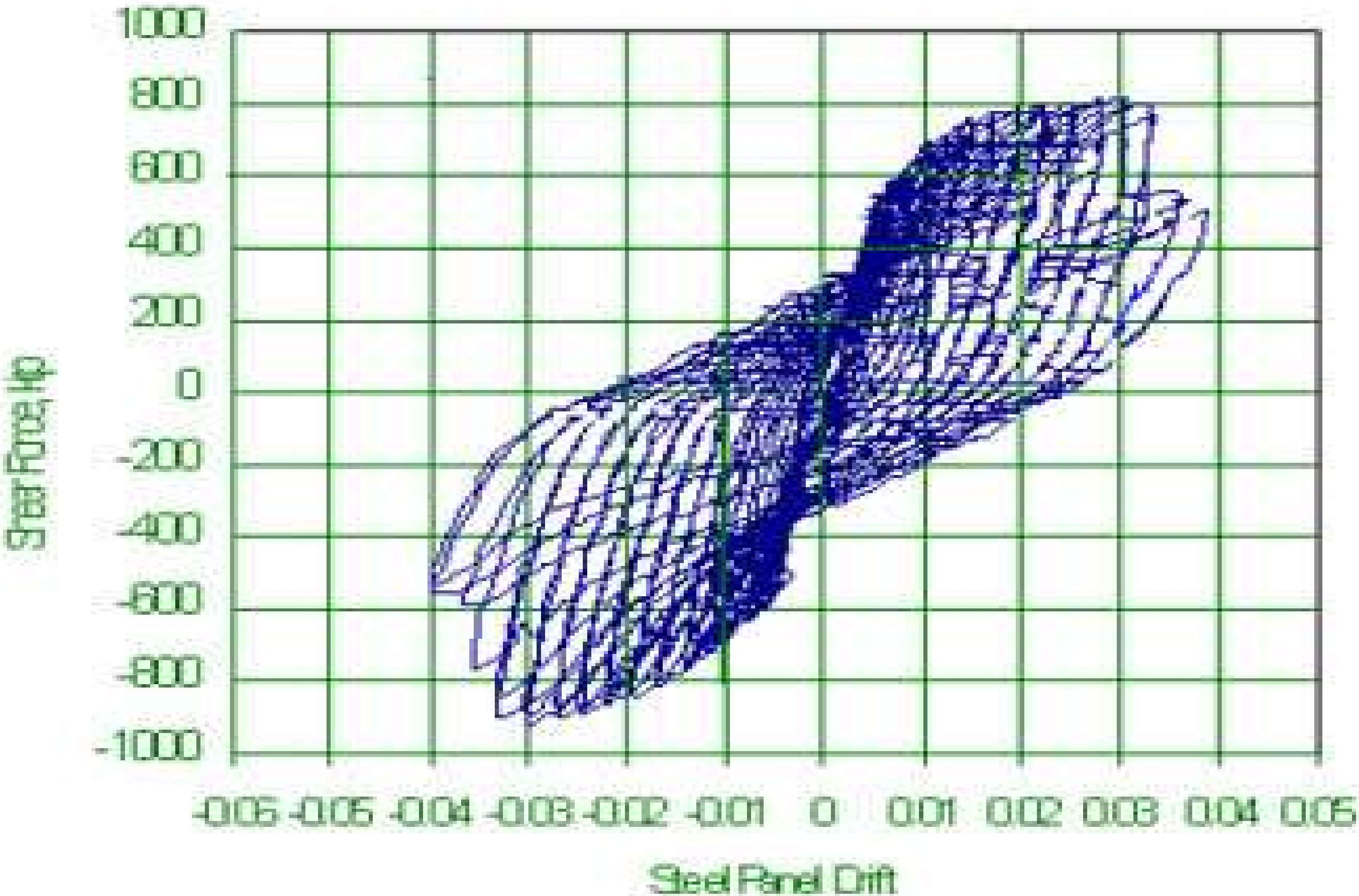


Test Specimen



Load vs. Displacement
(Curves From: Driver et al, 1996)

Force-Displacement Relation for Test 1-b and 1-c



رفتار دیوار برشی فولادی

- توربون وهمکاران در سال ۱۹۸۳ با توجه به سختی بالای اجزای مرزی قائم در مقایسه با بال های تیر ورق، مدل تحلیلی ساده ای را به منظور شبیه سازی رفتار میدان کششی بر اساس نظریه " میدان کششی خالص" توسعه دادند.
- در مدل تحلیلی مزبور که به " مدل نواری" نام گرفت میدان کشش توسط اعضای خرپایی کششی با زوایای شیب یکسان مدل میشود.
- به منظور انعکاس وجود میدان های کشش مخالف در بالا وپایین پانل فولادی مدل شده، تیرها درمدل نواری صلب فرض می شوند.
- حداقل ۱۰ عضو خرپایی در هر پانل برای بیان عمل میدان کششی کفایت می کند.

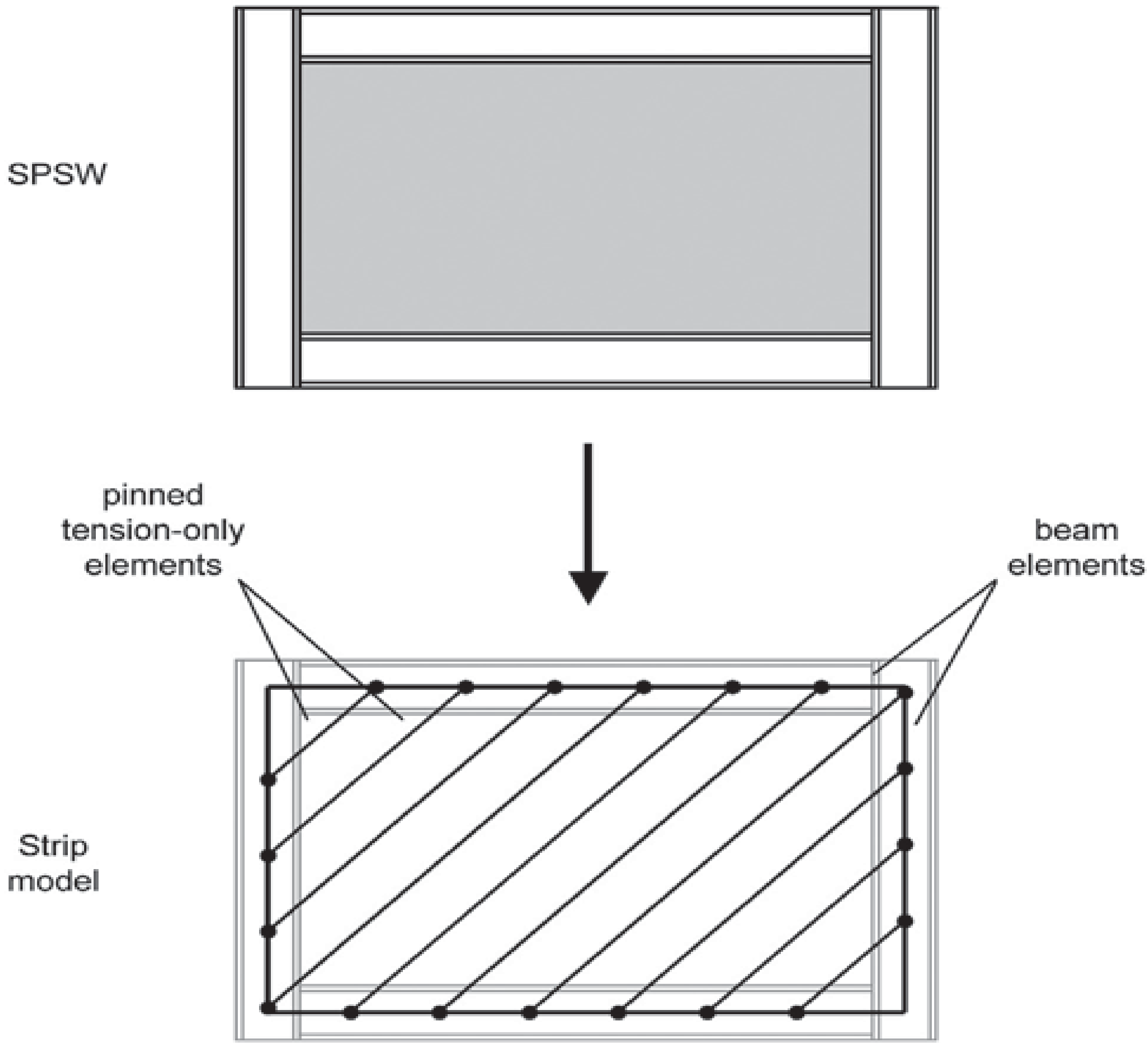
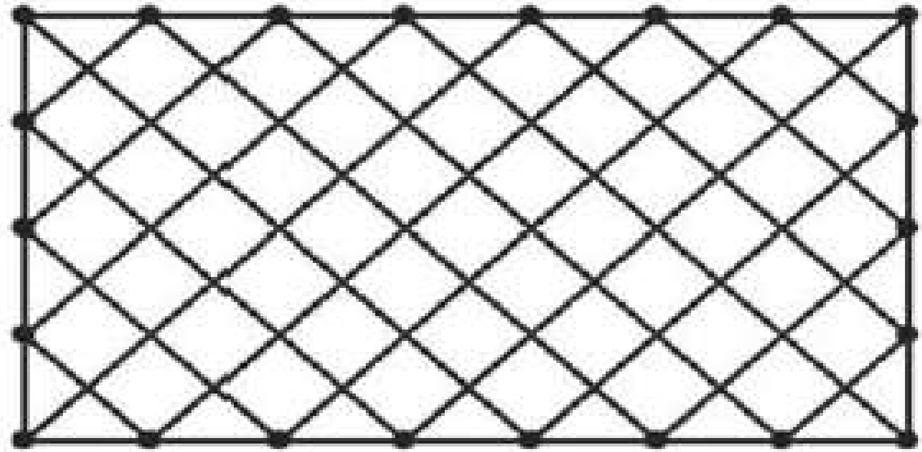
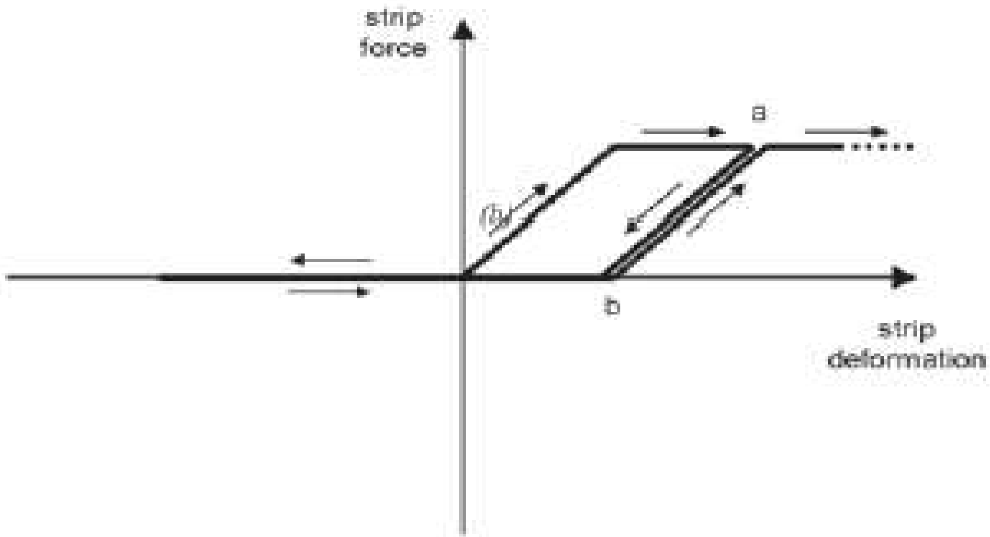


Fig. 2–38. Strip model for static (linear and nonlinear) analysis of SPW (courtesy of Diego López-García, Pontificia Universidad Católica de Chile, Chile).



(a)



(b)

Fig. 2–39. Strip model for cyclic static and dynamic nonlinear analysis: (a) diagram of panel model; (b) hysteretic strip force vs. strip deformation relationship (courtesy of Diego López-Garcia, Pontificia Universidad Católica de Chile, Chile).

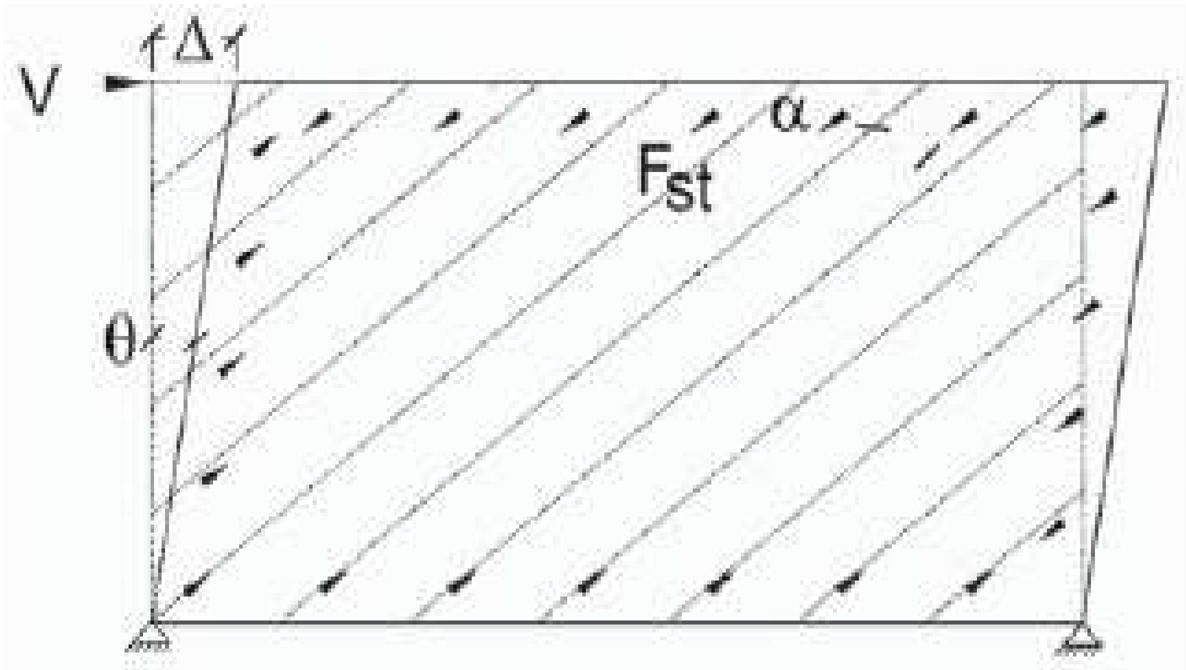


Fig. 2–42. Single-story collapse mechanism (Berman and Bruneau, 2003a).

- در این صورت حداکثر ظرفیت برشی دیوار برابر است با

$$V = \frac{1}{2} F_y t L \sin 2\alpha$$

- ودر صورتیکه اتصال تیر به ستون گیردار باشد، آنگاه V برابر است با:

$$V = \frac{1}{2} F_y t L \sin 2\alpha + \frac{4M_p}{h}$$

- که در آن Mp حداقل ظرفیت خمشی پلاستیک تیر یا ستون میباشد.
- رابطه فوق برای یک ساختمان چند طبقه چنین است:

$$\sum_{j=i}^{n} V_j = \frac{1}{2} F_y t_{wi} L \sin 2\alpha + \frac{4M_{pci}}{h_i}$$

- در رابطه فوق V_j نیروی جانبی اعمالی در تراز بالای طبقه i ام و t_{wi}، Mpci و h_i به ترتیب ضخامت ورق جان، لنگر پلاستیک ستون وارتفاع جان در طبقه i ام است.از رابطه فوق می توان ضخامت ورق جان دیوار برشی را تعیین کرد.

- استاندارد طراحی سازه های فولادی کانادا مدل نواری را به عنوان ابزاری برای طراحی دیوار برشی فولادی توصیه می کند.
- با استفاده از اصل حداقل انرژی، α زاویه شیب میدان کششی بر حسب سختی محوری اعضای مرزی به صورت رابطه زیر بدست می آید.

$$\tan^4 \alpha = \frac{1 + \dfrac{t_w L}{2Ac}}{1 + \dfrac{t_w h}{A\mathrm{b}}}$$

- که در آن رابطه t_w ضخامت دیوار، h ارتفاع طبقه، L عرض دهانه دیوار برشی وA_c وA_b به ترتیب مساحت مقطع اعضای مرزی قائم وافقی هستند.

- تیملر وکولاک در سال ۱۹۸۳ با منظور کردن سختی خمشی ستون ها ، زاویه شیب میدان کششی را به صورت تابعی از سختی محوری اعضای مرزی وسختی خمشی ستون ها چنین بدست آورده است.

$$\tan^4\alpha = \frac{1+\dfrac{t_w L}{2Ac}}{1+t_w h\left[\dfrac{1}{A_b}+\dfrac{h^3}{360 I_c L}\right]}$$

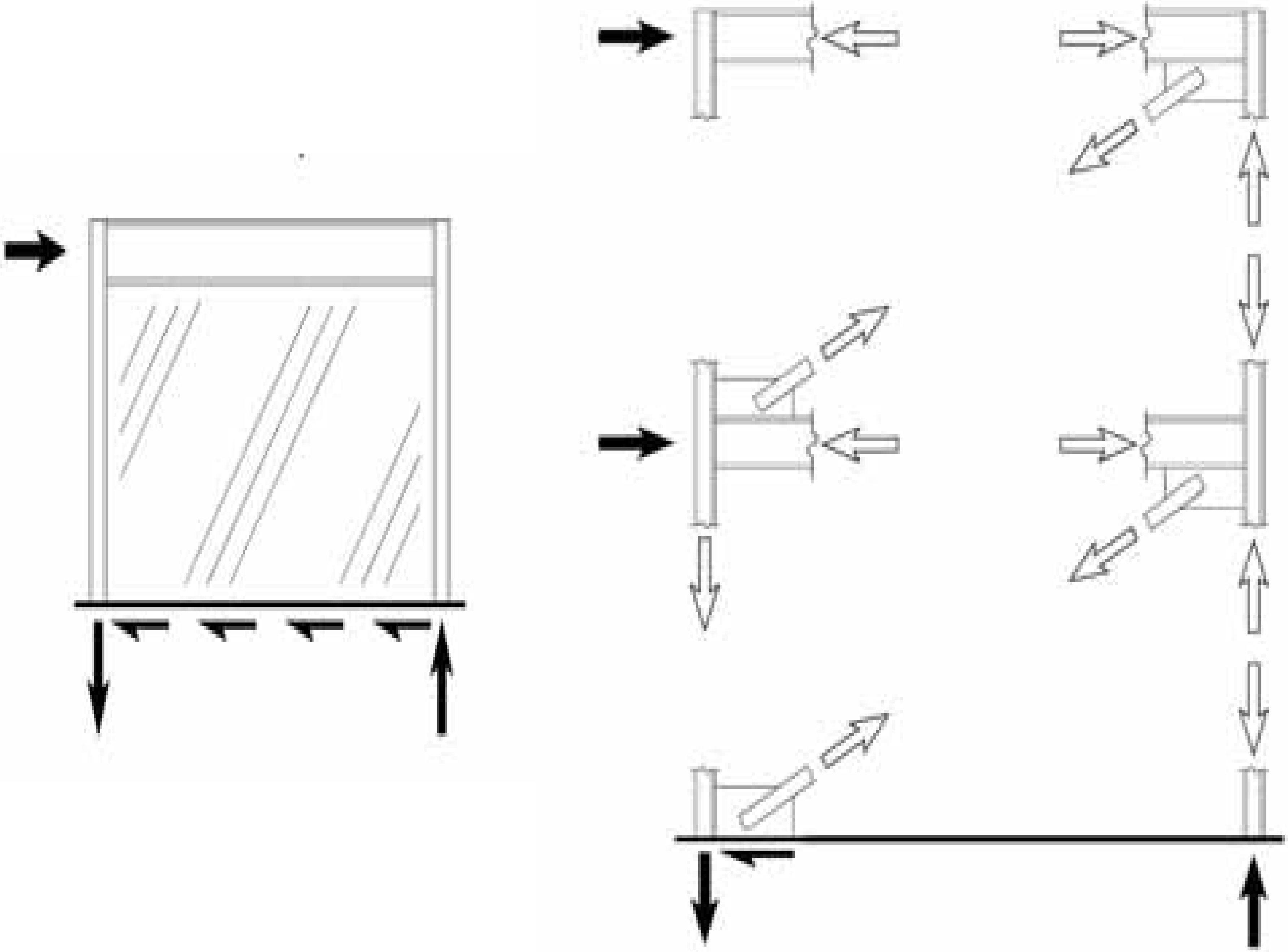
- برای توسعه تغییر شکلهای غیر ارتجاعی در ورق جان که نقش فیوز را در سیستم باربر جانبی لرزه ای بر عهده دارد،لازم است اعضای مرزی افقی (تیرها) وقائم (ستون ها) سختی کافی داشته باشند.از اینرو

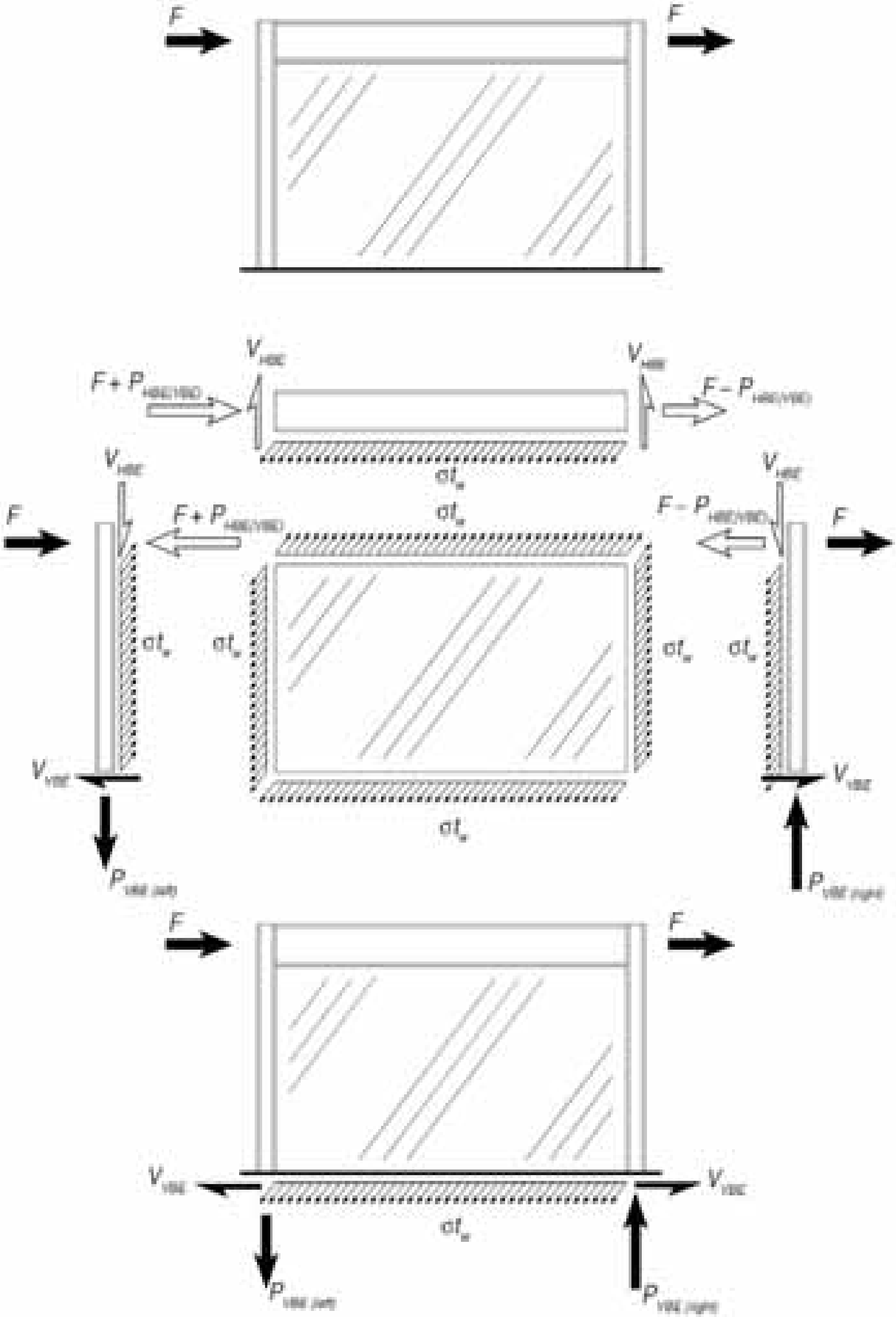
$$I_c \geq \frac{0.00307 t_w h^4}{L}$$

- I_c گشتاور اینرسی ستون حول محور عمود بر ورق جان می باشد.

رفتار دیوار برشی با ورق جان بدون سخت کننده

- آئین نامه طراحی AISC 341 استفاده از این سیستم را در مناطق لرزه خیزی متوسط وزیاد توصیه می نمایدوبرای مناطق با لرزه خیزی زیاد اتصال تیر به ستون را نیزگیردارتوصیه می نماید.
- در این سیستم ظرفیت فشاری ورق جان بسیار ناچیز است لیکن مقاومت کششی ورق جان که همراه با تجربه تغییر شکلهای فراارتجاعی بزرگ می باشد،زیاد است.
- رفتاردیوار برشی با ورق جان سخت نشده را می توان با یک قاب مهاربندی شده که اعضای قطری آن فقط به کشش کار میکنند، شبیه سازی نمود.
- تیرها تحت نیروی محوری فشاری ناشی از مولفه افقی مهاربندها، وستونها تحت نیروهای واژگونی ناشی از مولفه قائم مهاربندها هستند.
- اجزای قائم صلبیت کافی برای ایجاد میدان کششی در ورق جان را تأمین می کنند.





F = the applied lateral force on the wall

$P_{HBE(VBE)}$ = the axial force applied at the end of the HBE due to the web-plate tension on the VBE

P_{VBE} = the axial force reaction of the VBE

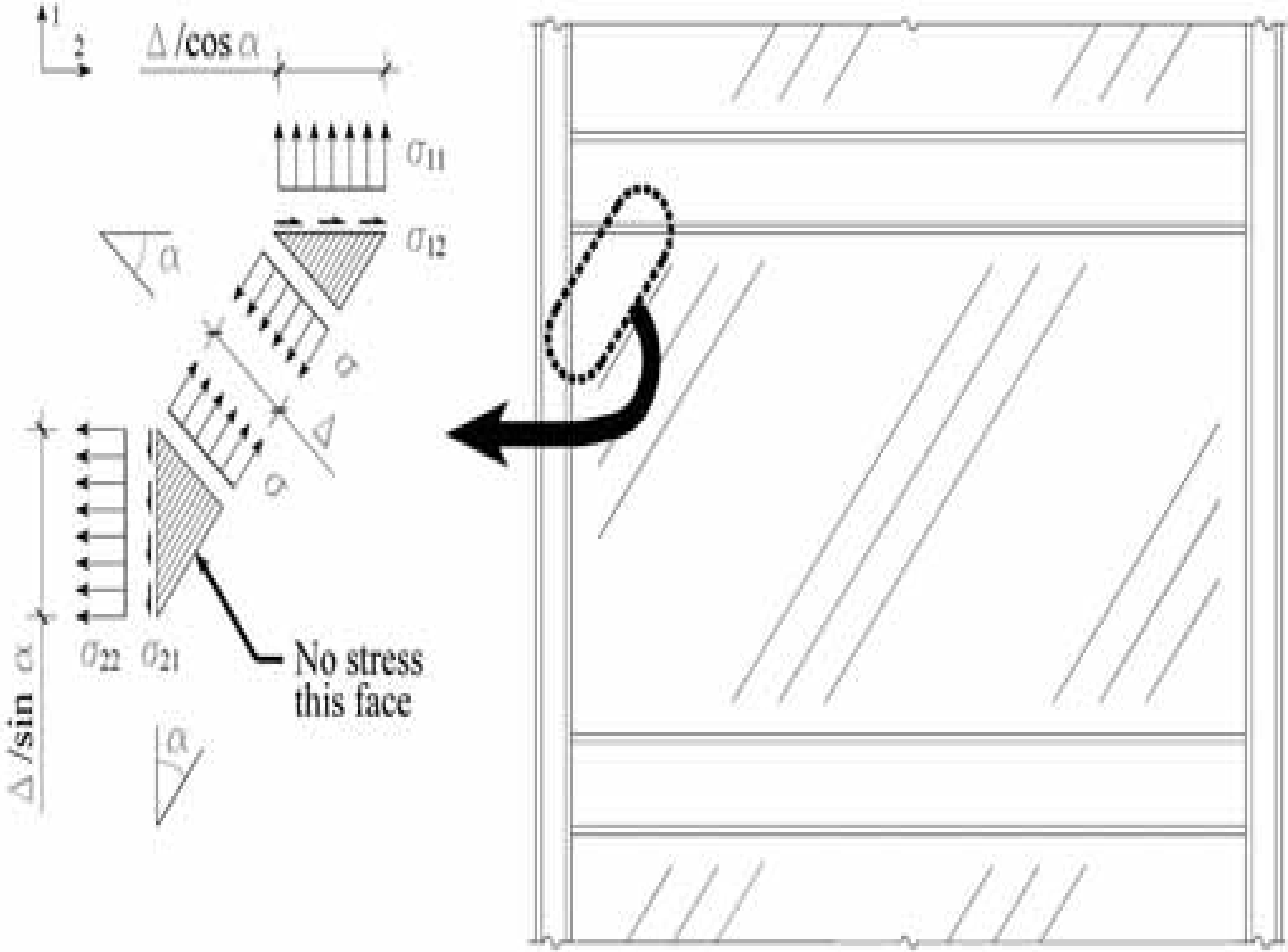
V_{HBE} = the shear reaction of the HBE due to the web-plate tension

V_{VBE} = the shear reaction of the VBE due to the web-plate tension

t_w = web-plate thickness

σ = web-plate tension stress

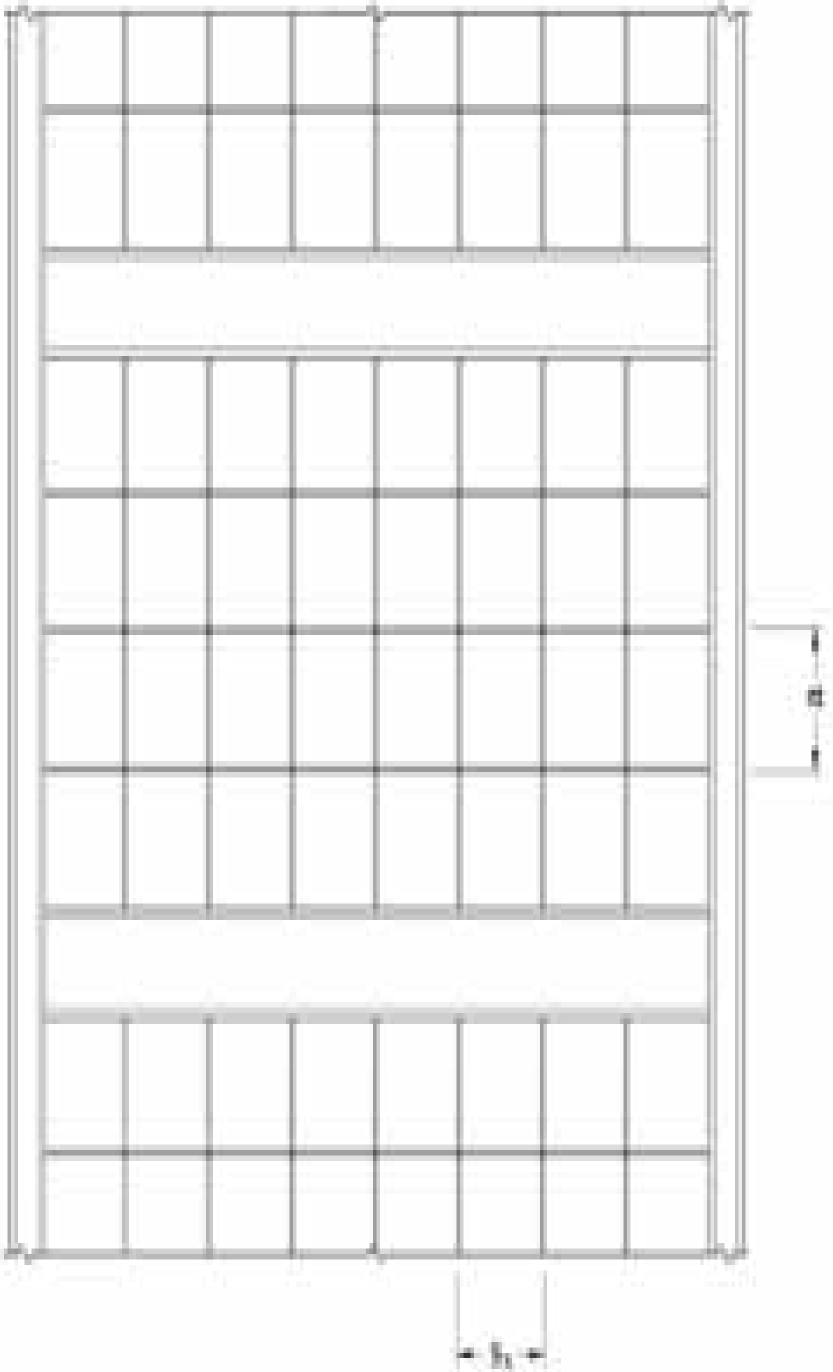
- مولفه های کششی ورق جان در قطعات میانی در اعضای مرزی افقی تقریباً یکدیگر را خنثی می کنند ولی درطبقات اول وآخر این اتفاق رخ نداده واستفاده از تیرهای قوی در این طبقات ضروری است.



1
2
Δ /cos α
σ11
σ12
α
σ
Δ
σ
σ22
σ21
No stress this face
Δ /sin α
α

رفتار دیوار برشی با ورق جان با سخت کننده

- دیوار برشی با ورق های سخت شده قادر خواهند بود نیروهای فشاری قابل ملاحظه ای را در ورق جان توسعه دهند. بنابراین اعضای مرزی تحت اثر خمش زیادی قرار ندارند.
- صلبیت ورق جان به اندازه ای است که نیروهای قابل ملاحظه ای در محل اتصال ورق به اجزای مرزی ایجاد نمی شود.
- اگر S فاصله بین سخت کننده ها باشد:



Where the spacing of stiffeners is equal in each direction, the limiting web slenderness ratio below which shear buckling is precluded is

$$\frac{s}{t_w} \leq 3.82\sqrt{\frac{E}{F_y}} \tag{3–4}$$

where

s = the spacing between stiffeners

t_w = the web-plate thickness

Where stiffeners are used in one direction only, the limiting web slenderness ratio is

$$\frac{s}{t_w} \leq 2.88\sqrt{\frac{E}{F_y}} \tag{3–5}$$

If the web plate is sufficiently stiffened to meet this criterion, its nominal strength is

$$V_n = 0.6F_y t_w L_{cf} \tag{3–6}$$

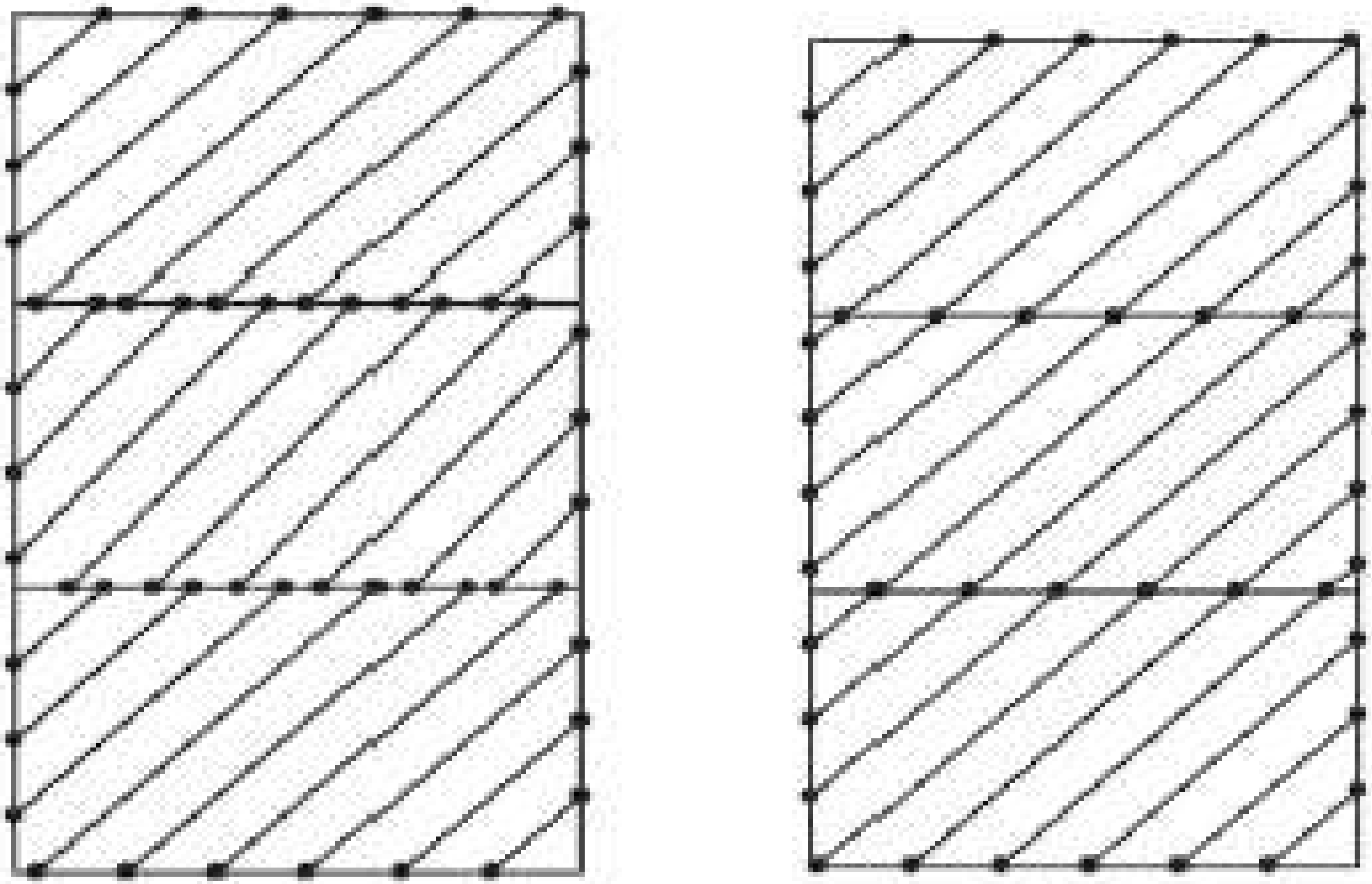
where

L_{cf} = the clear length of the web panel between VBE flanges

مدلهای تحلیل دیوارهای برشی فولادی

- ***مدل نواری***
- در این روش ورق جان توسط یک سری اعضای قطری موازی که فقط به کشش کار می کنند مدل می شود.
- حداقل ۱۰ المان نواری در هر پانل استفاده می شود.
- برای ساده سازی مدل می توان از میانگین زوایای میدان کششی پانل های دیوار برشی برای تمام ارتفاع سیستم استفاده نمود.
- در این صورت مساحت معادل هر نوار برابر است با

$$A_s = \frac{(L\cos\alpha + h\sin\alpha)t_w}{n}$$



(a) (b)

Fig. 2–41. Strip models having (a) staggered and (b) common strip nodes at the beams (courtesy of Diego López-García, Pontificia Universidad Católica de Chile, Chile).

- ***مدل غشایی ارتوتروپیک***
- در مدل المان غشایی با خواص ارتوتروپیک امکان اعمال مقاومت کششی وفشاری مجزا در دو جهت مختلف وجود دارد.
- خصوصیات ماده ای در امتداد زاویه α خواص واقعی ماده ودر جهت عمود بر آن با سختی صفر ویا مقادیر بسیار کم در نظر گرفته می شود.
- سختی برشی داخل صفحه المان نیز بایستی صفر شود.
- حداقل چهار المان در هر جهت توصیه شده است.

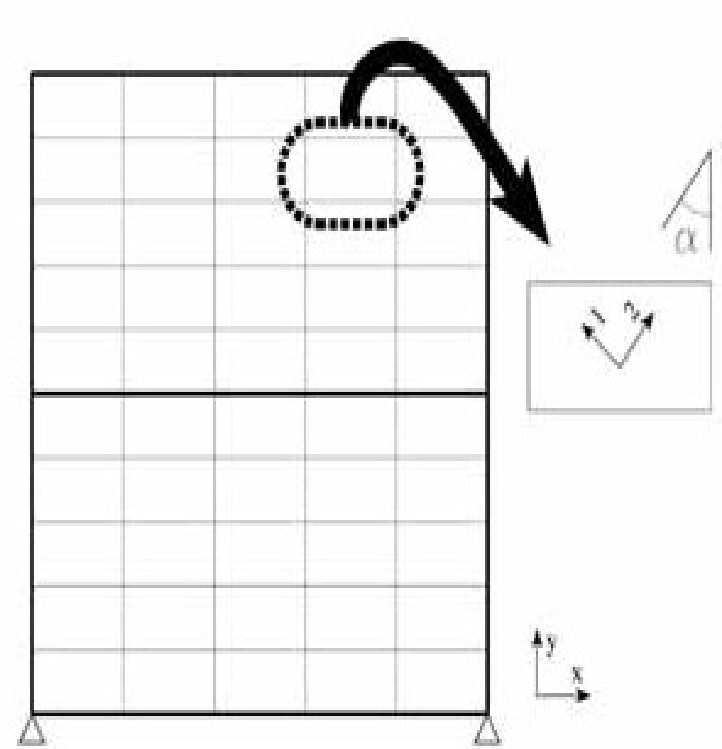


Table 4.1. Design Coefficients and Factors for Basic Seismic-force-resisting Systems
(The values in the table are those given by the IBC-2000)

Basic Seismic-force-resisting System	Response Modification Factor, R	System Over-Strength Factor Ω_o	Deflection Amplification Factor, C_d	System Limitations and Building Height Limitations (feet) by Seismic Design Category as Determined in Section 1616.3 of IBC-2000				
				A or B	C	D	E	F
Steel eccentrically braced frames, moment-resisting connections at columns away from links	8	2	4	NL	NL	160	160	100
Steel eccentrically braced frames, non-moment-resisting connections at columns away from links	7	2	4	NL	NL	160	160	100
Special steel concentrically braced frames	6	2	4	NL	NL	160	160	100
Ordinary steel concentrically braced frames	5	2	4 ½	NL	NL	160	160	100
Special reinforced concrete shear walls	6	2	5	NL	NL	160	160	100
Composite eccentrically braced frames	8	2	4	NL	NL	160	160	100
Special composite reinforced concrete shear walls with steel elements	6	2.5	5	NL	NL	160	160	100
Special steel moment frames	8	3	5.5	NL	NL	NL	NL	NL
Special reinforced concrete moment frames	8	3	5.5	NL	NL	NL	NL	NL
Dual system with special moment frames and steel eccentrically braced frames, moment-resisting connections, at columns away from links	8	2.5	4	NL	NL	NL	NL	NL
Dual system with special moment frames and steel eccentric braced frames, -moment-resisting connections, at columns away from links	8	2.5	4	NL	NL	NL	NL	NL
Dual system with special moment frames and special steel concentrically braced frames	8	2.5	6.5	NL	NL	NL	NL	NL
Dual system with special moment frames and special reinforced concrete shear walls	8	2.5	6.5	NL	NL	NL	NL	NL
Dual system with special moment frames and composite steel plate shear walls	8	2.5	6.5	NL	NL	NL	NL	NL

Notes: 1. This table only shows few systems and should not be used in actual design. For design refer to Table 1617.6 of IBC-2000.

2. NL=No Limit

Table 4.2. Proposed Design Coefficients and Factors for *Steel Shear Wall* Seismic-force-resisting systems (The author A. Astaneh-Asl tentatively proposed the values in the table)

Basic Seismic-force-resisting System	Response Modification Factor, R	System Over-Strength Factor Ω_o	Deflection Amplification Factor, C_d	System Limitations and Building Height Limitations (feet) by Seismic Design Category as Determined in Section 1616.3 of IBC-2000				
				A or B	C	D	E	F
1. *Un-stiffened* steel plate shear walls inside a gravity carrying steel frame with simple beam to column connections	6.5	2	5	NL	NL	160	160	100
2. *Stiffened* steel plate shear walls inside a gravity carrying steel frame with simple beam-to-column connections	7.0	2	5	NL	NL	160	160	160
3. Dual system with special steel moment frames and *un-stiffened* steel plate shear walls	8	2.5	4	NL	NL	NL	NL	NL
4. Dual system with special steel moment frames and *stiffened* steel plate shear walls	8.5	2.5	4	NL	NL	NL	NL	NL

Note: NL=No Limit

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b
7. Steel and concrete composite plate shear walls	14.3	7½	2½	6
8. Steel and concrete composite special shear walls	14.3	7	2½	6
9. Steel and concrete composite ordinary shear walls	14.3	6	2½	5
10. Special reinforced masonry shear walls	14.4	5½	3	5
11. Intermediate reinforced masonry shear walls	14.4	4	3	3½
12. Steel buckling-restrained braced frames	14.1	8	2½	5
13. Steel special plate shear walls	14.1	8	2½	6½

Failure modes of steel plate wall

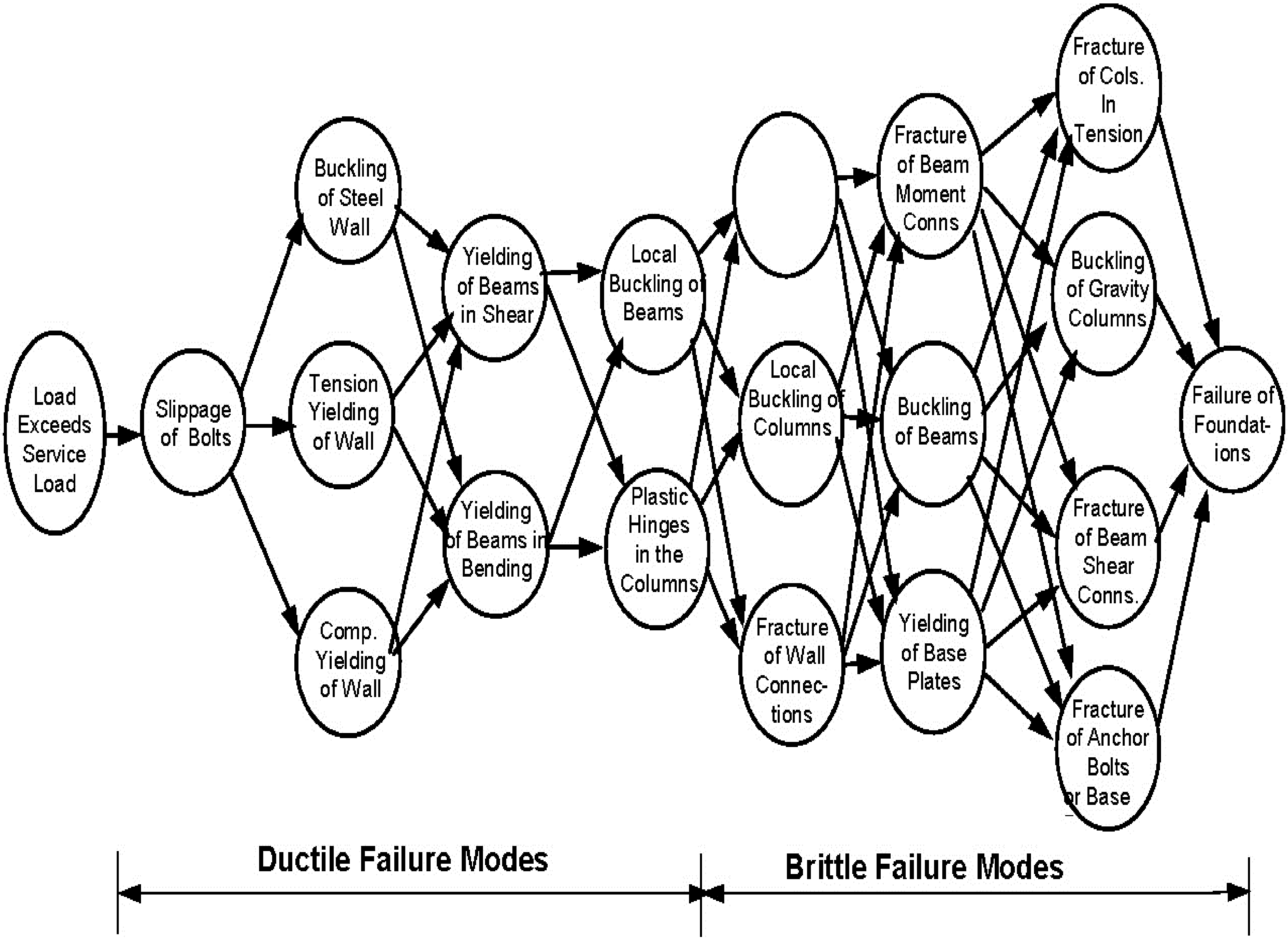
1. Slippage of bolts (ductile).
2. Buckling of the steel plate (ductile).
3. Yielding of the steel plate (ductile).
4. Fracture of wall plate (brittle).
5. Fracture of the connections of steel wall to boundary columns and beams (brittle).

Failure modes of top and bottom beams

6. Shear yielding of top and bottom beams (ductile).
7. Plastic hinge formation in top and bottom beams (ductile).
8. Local buckling in the top and bottom beam flanges or web (ductile if $b/t \leq \lambda_p$).
9. Fracture of moment connections of the beams in dual systems (brittle).
10. Overall or lateral-torsional buckling of beams (brittle).
11. Fracture of shear connections of beams (brittle).

Failure modes of Boundary Columns

12. Plastic hinge formation at the top and bottom of columns (ductile).
13. Local buckling of boundary columns (ductile if $b/t \leq \lambda_p$).
14. Overall buckling of boundary columns (ductile if $\lambda_c = (KL/\pi r)\sqrt{(F_y/E)} \leq 1.0$.)
15. Tension fracture of boundary columns or their splices (brittle).
16. Yielding of base plates of boundary columns in uplift (ductile)
17. Fracture of anchor bolts or base plates at the base of columns in uplift (brittle)
18. Fracture of column base plates in bending and/or uplift (brittle)
19. Failure of foundations of the wall (brittle).


Load Exceeds Service Load
Slippage of Bolts
Buckling of Steel Wall
Tension Yielding of Wall
Comp. Yielding of Wall
Yielding of Beams in Shear
Yielding of Beams in Bending
Local Buckling of Beams
Plastic Hinges in the Columns
Local Buckling of Columns
Fracture of Wall Connec-tions
Fracture of Beam Moment Conns
Buckling of Beams
Yielding of Base Plates
Fracture of Cols. In Tension
Buckling of Gravity Columns
Fracture of Beam Shear Conns.
Fracture of Anchor Bolts or Base
Failure of Foundat-ions
Ductile Failure Modes
Brittle Failure Modes

SPECIAL PLATE SHEAR WALLS (SPSW) AISC-341

- **1. Scope**
- *In special plate shear walls (SPSW), the slender unstiffened steel plates (webs) connected to surrounding horizontal and vertical boundary elements (HBE and VBE) are designed to yield and behave in a ductile hysteretic manner during earthquakes (see Figure C-F5.1). All HBE are also rigidly connected to the VBE with moment resisting connections able to develop the expected plastic moment of the HBE. Each web must be surrounded by boundary elements.*
- **2. Basis of Design**
- *Yielding of the webs occurs by development of tension field action at an angle close to 45° from the vertical, and buckling of the plate in the orthogonal direction. Past research shows that the sizing of VBE and HBE in a SPSW makes it possible to develop this tension field action across all of the webs.*

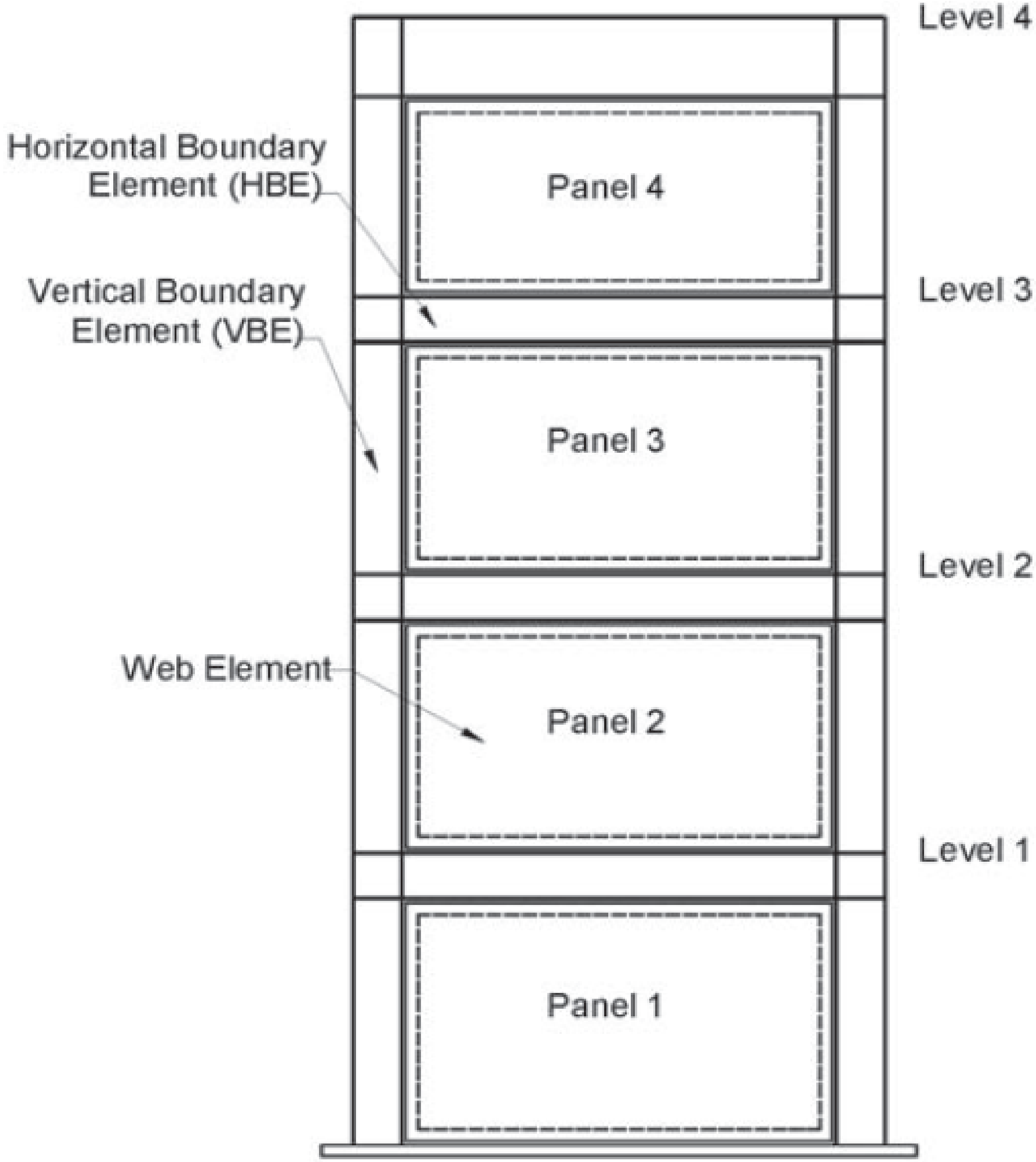


Fig. C-F5.1. Schematic of special plate shear wall.

- *Except for cases with very stiff HBE and VBE, yielding in the webs develops in a progressive manner across each panel. Because the webs do not yield in compression, continued yielding upon repeated cycles of loading is contingent upon the SPSW being subjected to progressively larger drifts, except for the contribution of plastic hinging developing in the HBE to the total system hysteretic energy. In past research (Driver et al., 1997), the yielding of boundary elements contributed approximately 25 to 30% of the total load strength of the system.*
- *With the exception of plastic hinging at the ends of HBEs, the surrounding HBEs and VBEs are designed to remain essentially elastic when the webs are fully yielded. Plastic hinging at the ends of HBEs is needed to develop the plastic collapse mechanism of this system. Plastic hinging in the middle of HBEs, which could partly prevent yielding of the webs, is deemed undesirable. Cases of both desirable and undesirable yielding in VBE have been observed in past testing.*

- *Research literature often compares the behavior of steel plate walls to that of a vertical plate girder, indicating that the webs of a SPSW resist shears by tension field action and that the VBE of a SPSW resist overturning moments. While this analogy is useful in providing a conceptual understanding of the behavior of SPSW, many significant differences exist in the behavior and strength of the two systems. Past research shows that the use of structural shapes for the VBE and HBE in SPSW (as well as other dimensions and details germane to SPSW) favorably impacts orientation of the angle of development of the tension field action, and makes possible the use of very slender webs (having negligible diagonal compressive strength). Sizeable top and bottom HBEs are also required in the SPSW to anchor the significant tension fields that develop at the ends of the structural system. Limits imposed on the maximum web slenderness of plate girders to prevent flange buckling, or due to transportation requirements, are also not applicable to SPSW which are constructed differently. For these reasons, the use of beam design provisions in the Specification for the design of SPSW is not appropriate (Berman and Bruneau, 2004).*

- **3. Analysis**
- *The webs of SPSW shall not be considered as resisting gravity forces.*
- *The required strength of HBEs, vertical boundary elements (VBEs), and connections in SPSW shall be based on the **load combinations in the applicable building code that include the amplified seismic load**.*
- *In determining the amplified seismic load the effect of horizontal forces including overstrength ,E_{mh} ,shall be determined from an analysis in which all webs are assumed to resist forces corresponding to their expected strength in tension at an angle, α, as determined in Section F5.5b and HBE are resisting flexural forces at each end equal to **$1.1R_yM_p$ (LRFD).***
- *Webs shall be determined to be in tension neglecting the effects of gravity loads.*
- *The expected web yield stress shall be taken as R_yF_y . When perforated walls are used, the effective expected tension stress is as defined in Section F5.7a(4).*

- *Per capacity design principles,* ***all edge boundary elements (HBE and VBE) shall be designed to resist the maximum forces developed by the tension field action of the webs fully yielding****. Axial forces, shears and moments develop in the boundary elements of the SPSW as a result of the response of the system to the overall overturning and shear, and this tension field action in the webs.* ***Actual web thickness must be considered for this calculation****, because webs thicker than required may have to be used due to availability, or minimum thickness required for welding.*
- ***At the top panel of the wall, the vertical components of the tension field shall be anchored to the HBE****. The HBE shall have sufficient strength to allow development of full tensile yielding across the panel width.*
- ***At the bottom panel of the wall, the vertical components of the tension field shall also be anchored to the HBE****. The HBE shall have sufficient strength to allow development of full tensile yielding across the panel width. This may be accomplished by continuously anchoring the HBE to the foundation.*

- *For intermediate HBE of the wall, the anticipated variation between the top and bottom web normal stresses acting on the HBE is usually small, or null when webs in the panel above and below the HBE have identical thickness. While top and bottom HBE are typically of substantial size, intermediate HBE are relatively smaller.*
- ***For the design of HBE, it may be important to recognize the effect of vertical stresses introduced by the tension field forces in reducing the plastic moment of the HBE****. Concurrently, free-body diagrams of HBEs should account for the additional shear and moments introduced by the eccentricity of the horizontal component of the tension fields acting at the top and bottom of the HBEs.*
- *Beyond plastic-hinge formation at the ends of the HBE, in some instances the engineer may be able to justify yielding of the boundary elements by demonstrating that the yielding of a particular edge boundary element will not cause reduction on the SPSW shear capacity to support the demand and will not cause a failure in vertical gravity carrying capacity.*

- *Forces and moments in the members (and connections), including those resulting from tension field action, may be determined from a plane frame analysis. The web is represented by a series of inclined pin-ended strips, as described in Commentary Section F5.5b. A minimum of ten equally spaced pin-ended strips per panel will be used in such an analysis.*
- *A number of analytical approaches are possible to achieve capacity design and determine the same forces acting on the vertical boundary elements. Some example methods applicable to SPSW follow. In all cases, actual web thickness must be considered, for reasons described earlier.*
- ***Nonlinear push-over analysis****. A model of the SPSW can be constructed in which bilinear elasto-plastic web elements of strength $R_y F_y A_s$ are introduced in the direction α. Bilinear plastic hinges can also be introduced at the ends of the horizontal boundary elements. Standard push-over analysis conducted with this model will provide axial forces, shears and moments in the boundary frame when the webs develop yielding. Separate checks are required to verify that plastic hinges do not develop in the horizontal boundary elements, except at their ends.*

- ***Indirect capacity design approach***. *The Canadian Standards Association Standard ,Limit States Design of Steel Structures (CSA, 2001), proposes that loads in the vertical boundary members can be determined from the gravity loads combined with the seismic loads increased by the amplification factor,*
- $B = V_e / V_u$
- *where*
- V_e *= expected shear strength, at the base of the wall, determined for the web thickness supplied, kips*
- $= 0.5\, R_y\, F_y t_w\, L \sin 2\alpha$
- V_u *= factored lateral seismic force at the base of the wall In determining the loads in VBEs, the amplification factor, B, need not be taken as greater than R.*
- *The VBE design axial forces shall be determined from overturning moments defined as follows:*

- *- The moment at the base is BM_u , where M_u is the factored seismic overturning moment at the base of the wall corresponding to the force Vu*
- *- The moment BM_u extends for a height H but not less than two stories from the base*
- *- The moment decreases linearly above a height H to B times the overturning moment at one story below the top of the wall, but need not exceed R times the factored seismic overturning moment at the story under consideration corresponding to the force V_u*
- *The local bending moments in the VBE due to tension field action in the web shall be multiplied by the amplification factor B.*

- **4. System Requirements**
- **4a. Stiffness of Boundary Elements**
- *The vertical boundary elements (VBEs) shall have moments of inertia about an axis taken perpendicular to the plane of the web, **I_c, not less than $0.0031 t_w h^4/L$**.*
- *The horizontal boundary elements (HBEs) shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_b, **not less than $0.0031L^4/h$ times the difference in web plate thicknesses above and below,***
- *Where*
- *I_b = moment of inertia of a HBE taken perpendicular to the direction of the web plate line, in.4 (mm4)*
- *I_c = moment of inertia of a VBE taken perpendicular to the direction of the web plate line, in.4 (mm4)*
- *L = distance between VBE centerlines, in. (mm)*
- *h = distance between HBE centerlines, in. (mm)*
- *t_w = thickness of the web, in. (mm)*

- **4b. HBE-to-VBE Connection Moment Ratio**
- *The moment ratio provisions in Section E3.4a shall be met for all HBE/VBE intersections without consideration of the effects of the webs.*
- **4c. Bracing**
- *HBE shall be braced to satisfy the requirements for moderately ductile members in Section D1.2a.*
- **4d. Openings in Webs**
- *Openings in webs shall be bounded on all sides by intermediate boundary elements extending the full width and height of the panel respectively, unless otherwise justified by testing and analysis or permitted by Section F5.7.*
- **5. Members**
- **5a. Basic Requirements**
- *HBE, VBE and intermediate boundary elements shall satisfy the requirements of Section D1.1 for highly ductile members.*

- **5b. Webs**
- *The panel design shear strength, φV_n (LRFD), in accordance with the limit state of shear yielding, shall be determined as follows:*
- $V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha$, $\varphi = 0.90$
- *where*
- L_{cf} *= clear distance between column flanges, in. (mm)*
- t_w *= thickness of the web, in. (mm)*
- α *=angle of web yielding in degrees, as measured relative to the vertical. The angle of inclination, α, is permitted to be taken as 40°, or is permitted to be calculated as follows:*

$$\tan^4 \alpha = \frac{1 + \dfrac{t_w L}{2Ac}}{1 + t_w h\left[\dfrac{1}{A_b} + \dfrac{h^3}{360 I_c L}\right]}$$

- where
- A_b *= cross-sectional area of an HBE, in.2 (mm2)*
- A_c *= cross-sectional area of a VBE, in.2 (mm2)*

- **5c. Protected Zone**
- *The protected zone of SPSW shall satisfy Section D1.3 and include the following:*
- ***(1) The webs of SPSW***
- ***(2) Elements that connect webs to HBEs and VBEs***
- ***(3) The plastic hinging zones at each end of HBEs, over a region ranging from the face of the column to one beam depth beyond the face of the column, or as otherwise specified in Section E3.5c***

- **6. Connections**
- **6a. Demand Critical Welds**
- *The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:*
- ***(1) Groove welds at column splices***
- ***(2) Welds at column-to-base plate connections***
- *Exception: Where it can be shown that column hinging at, or near, the base plate is precluded by conditions of restraint, and in the absence of net tension under load combinations including the amplified seismic load, demand critical welds are not required.*
- ***(3) Welds at HBE-to-VBE connections***

- **6b. HBE-to-VBE Connections**
- *HBE-to-VBE connections shall satisfy the requirements of Section E1.6b.*
- ***(1) Required Strength***
- *The **required shear strength** of an HBE-to-VBE connection shall be based on the load combinations in the applicable building code that include the amplified seismic load. In determining the amplified seismic load, the effect of horizontal forces includin overstrength, E_{mh} , shall be taken as the shear calculated from Equation*
- $E_{mh} = 2[1.1R_yM_p]/L_{cf}$
- *together with the shear resulting from the expected yield strength in tension of the webs yielding at an angle α.*
- ***(2) Panel Zones***
- *The VBE panel zone next to the top and base HBE of the SPSW shall comply with the requirements in Section E3.6e.(1) Groove welds at column splices*

- **6c. Connections of Webs to Boundary Elements**
- *The required strength of web connections to the surrounding HBE and VBE shall equal the* ***expected yield strength, in tension, of the web calculated at an angle α****.*
- **6d. Column Splices**
- *Column splices shall comply with the requirements of Section D2.5. Where welds are used to make the splice, they shall be* ***complete-joint-penetration groove welds****.*
- *Column splices shall be designed to develop at* ***least 50% of the lesser available flexural strength of the connected members.*** *The required shear strength, V_u , shall be determined by Equations F4-2a .*

- # 7. Perforated Webs
- ## 7a. Regular Layout of Circular Perforations
- *A perforated plate conforming to this section is permitted to be used as the web of an SPSW. Perforated webs shall have a regular pattern of holes of uniform diameter spaced evenly over the entire web-plate area in an array pattern so that holes align diagonally at a uniform angle to vertical. Edges of openings shall have a surface roughness of 500 μ-in. (13 microns) or less.*
- ### (1) Strength
- *The panel design shear strength, φV_n (LRFD) , in accordance with the limit state of shear yielding, shall be determined as follows for perforated webs:*

$$V_n = 0.42 F_y t_w L_{cf} \left(1 - \frac{0.7D}{S_{diag}} \right) \qquad \text{(F5-3)}$$

- *φ = 0.90 (LRFD)*

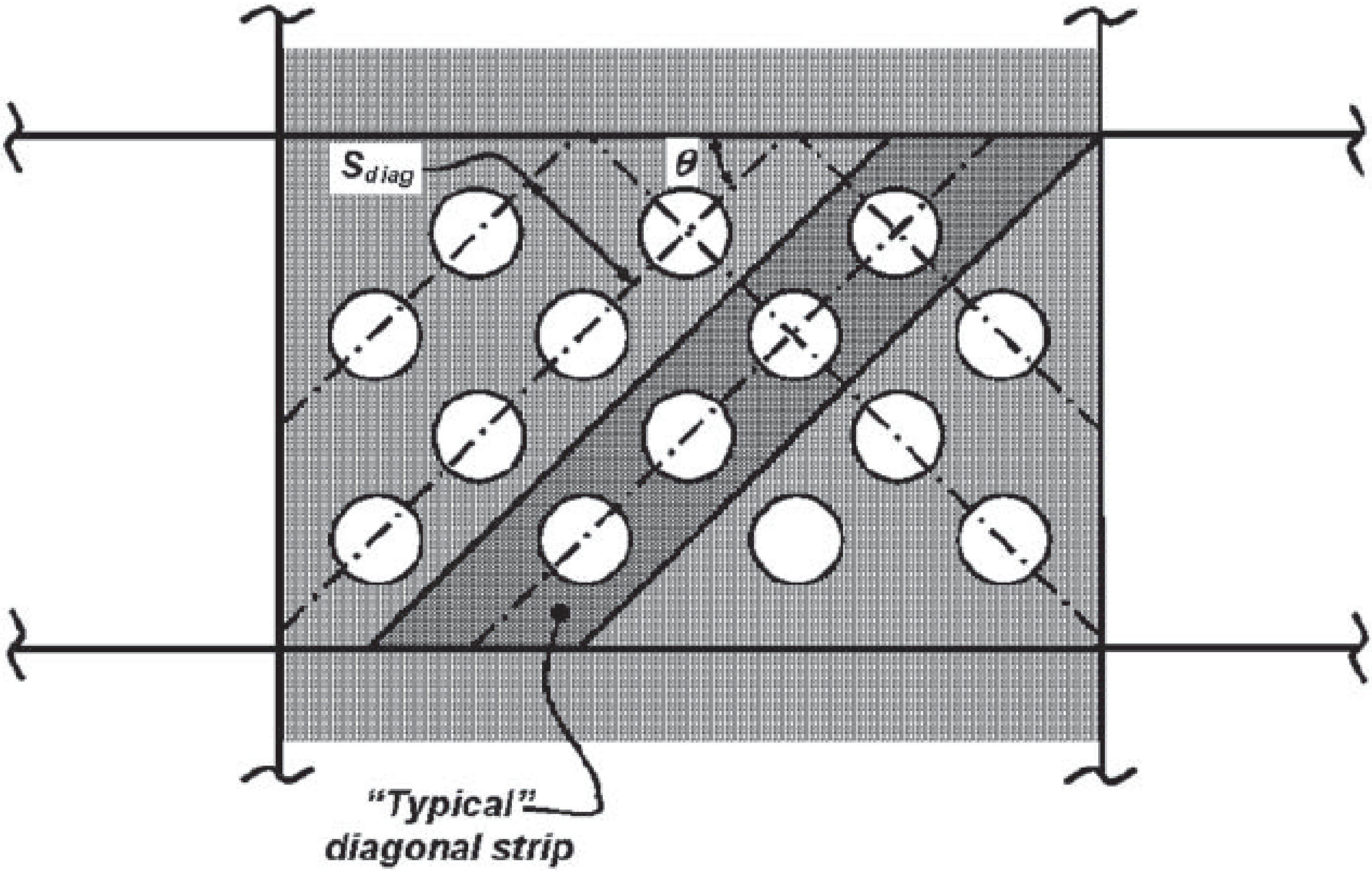


Fig. C-F5.7. Schematic detail of special perforated steel plate wall and typical diagonal strip.

- *where*
- *D = diameter of the holes, in. (mm)*
- S_{diag} *= shortest center-to-center distance between the holes, in. (mm)*
- **(2) Spacing**
- *The spacing,* S_{diag} *, shall be at least 1.67D.*
- *The distance between the first holes and web connections to the HBEs and VBEs shall be at least D, but shall not exceed (D + 0.7*S_{diag}*).*
- **(3) Stiffness**
- *The stiffness of such regularly perforated infill plates shall be calculated using an effective web-plate thickness,* t_{eff} *, given by:*

$$t_{eff} = \frac{1 - \frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)}{1 - \frac{\pi}{4}\left(\frac{D}{S_{diag}}\right)\left(1 - \frac{N_r D Sin\alpha}{H_c}\right)} t_w \qquad \text{(F5-4)}$$

- *where*

- *H_c = clear column (and web-plate) height between beam flanges, in. (mm)*
- *N_r = number of horizontal rows of perforations*
- *t_w = web-plate thickness, in. (mm)*
- *α = angle of the shortest center-to-center lines in the opening array to vertical, degrees*
- **(4) Effective Expected Tension Stress**
- The effective expected tension stress to be used in place of the effective tension stress for analysis per Section F5.3 is $R_y F_y (1 - 0.7\,D/S_{diag})$.
- **7b. Reinforced Corner Cut-Out**
- *Quarter-circular cut-outs are permitted at the corners of the webs provided that the webs are connected to a reinforcement arching plate following the edge of the cutouts. The plates shall be designed to allow development of the full strength of the solid web and maintain its resistance when subjected to deformations corresponding to the design story drift. This is deemed to be achieved if the following conditions are met.*

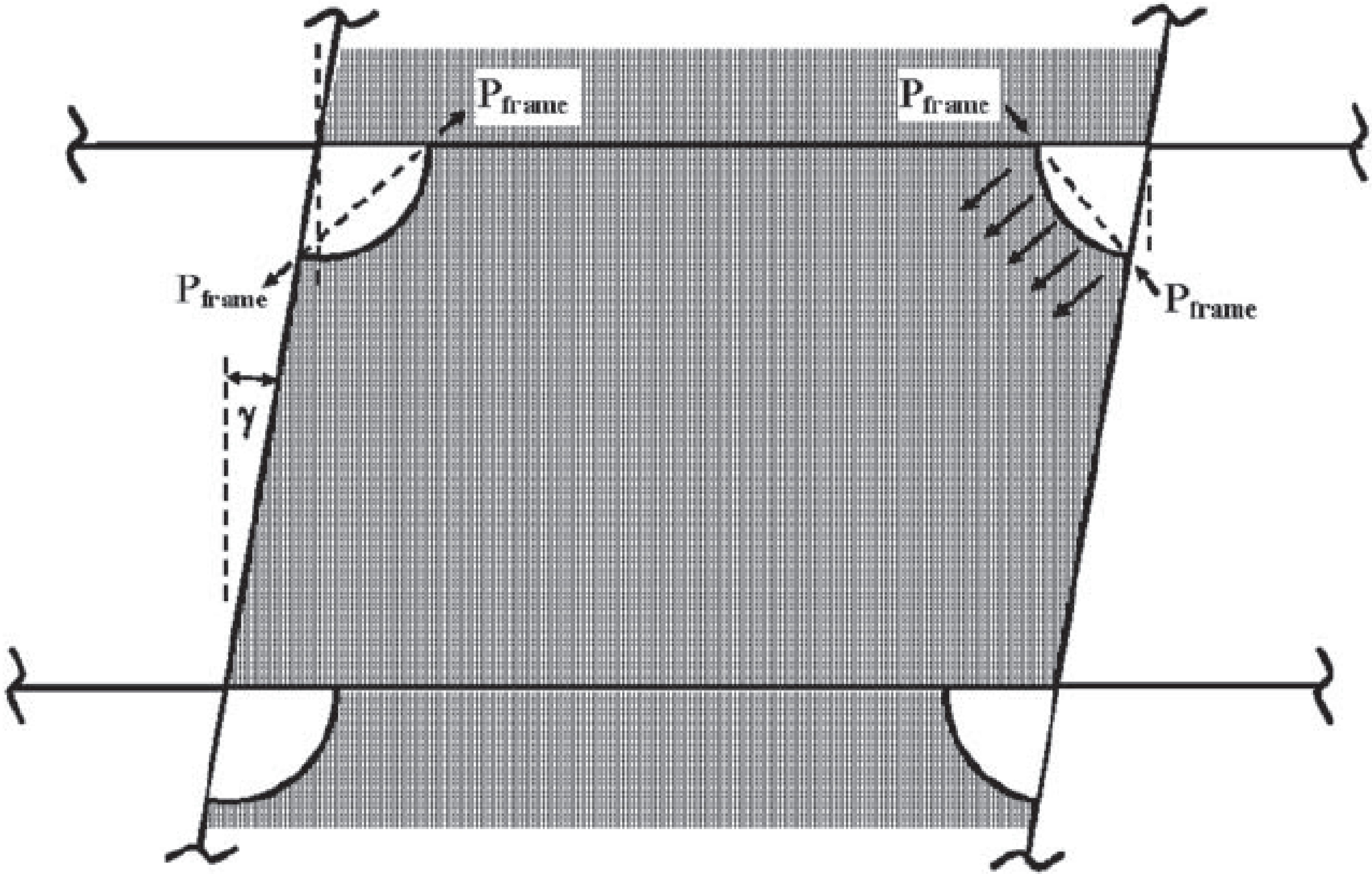


Fig. C-F5.8. Arch end reactions due to frame deformations, and infill panel forces on arches due to tension field action on reinforced cut-out corner.

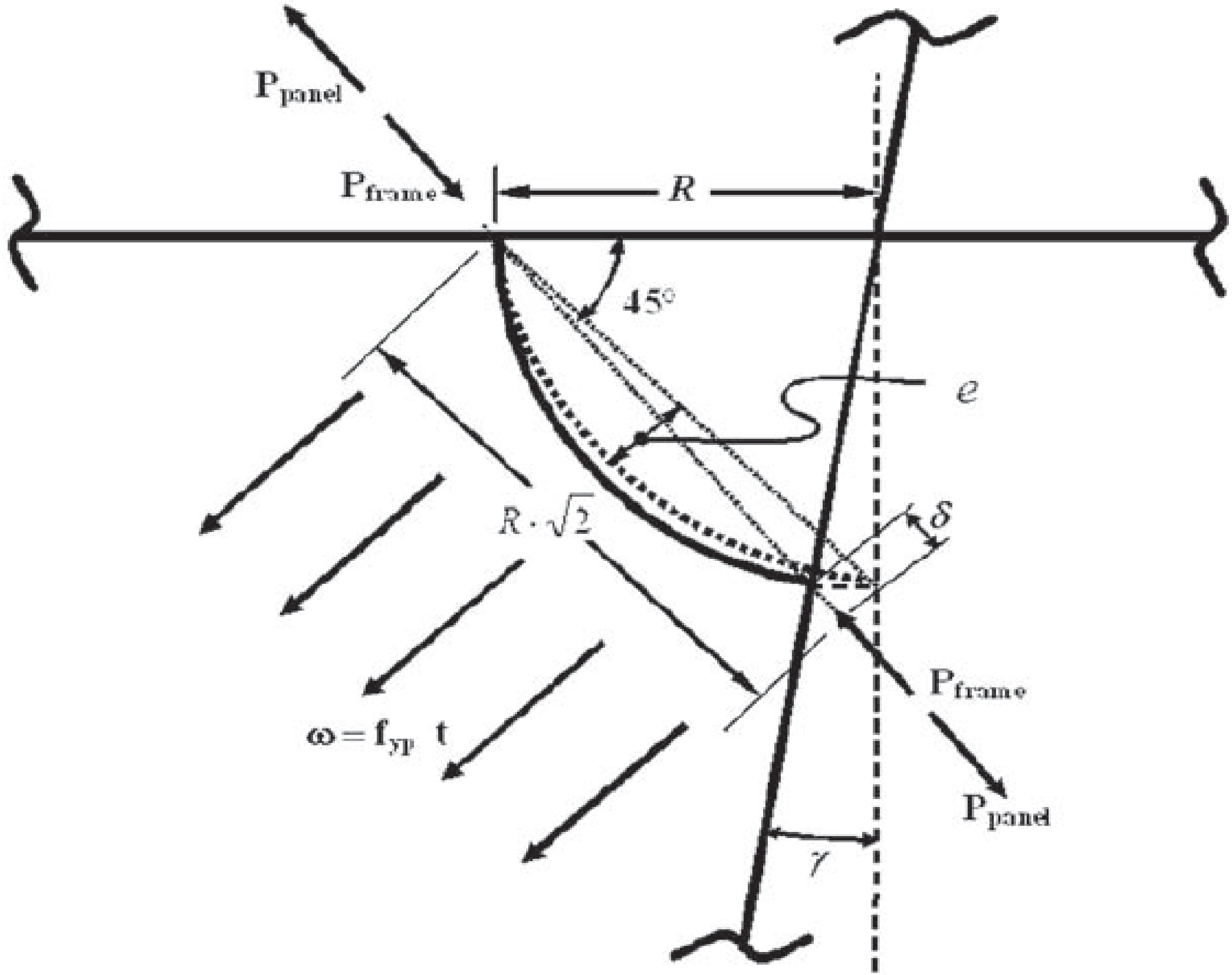


Fig. C-F5.9. Deformed configurations and forces acting on right arch.

- **(1) Design for Tension**
- *The arching plate shall have the available strength to resist the axial tension force resulting from web-plate tension in the absence of other forces.*

$$P_u = \frac{R_y F_y t_w R^2}{4e} \qquad \text{(F5-5)}$$

- *as appropriate,*
- *where*
- *R = radius of the cut-out, in. (mm)*
- *Ry = ratio of the expected yield stress to the specified minimum yield stress*
- $e = R\left(1-\sqrt{2}/2\right)$ *, in. (mm)* *(F5-6)*
- *HBEs and VBEs shall be designed to resist the tension axial forces acting at the end of the arching reinforcement.*

- **(2) Design for Beam-to-Column Connection Forces**
- *The arching plate shall have the available strength to resist the combined effects of axial force and moment in the plane of the web resulting from connection in the absence of other forces. These forces are:*

$$P_u = \frac{15EI_y}{16e^2}\left(\frac{\Delta}{H}\right) \qquad (F5\text{-}7)$$

- *as appropriate.*
- *The moments are:*

$$M_u = P_u e \qquad (F5\text{-}8)$$

- *as appropriate,*
- *where*
- *E = modulus of elasticity, ksi (MPa)*
- *H = height of story, in. (mm)*
- *I_y = moment of inertia of the plate about the y-axis, in.^{4} (mm^4)*
- *Δ = design story drift, in. (mm)*

Behavior of Steel Shear Walls Under Applied Shear

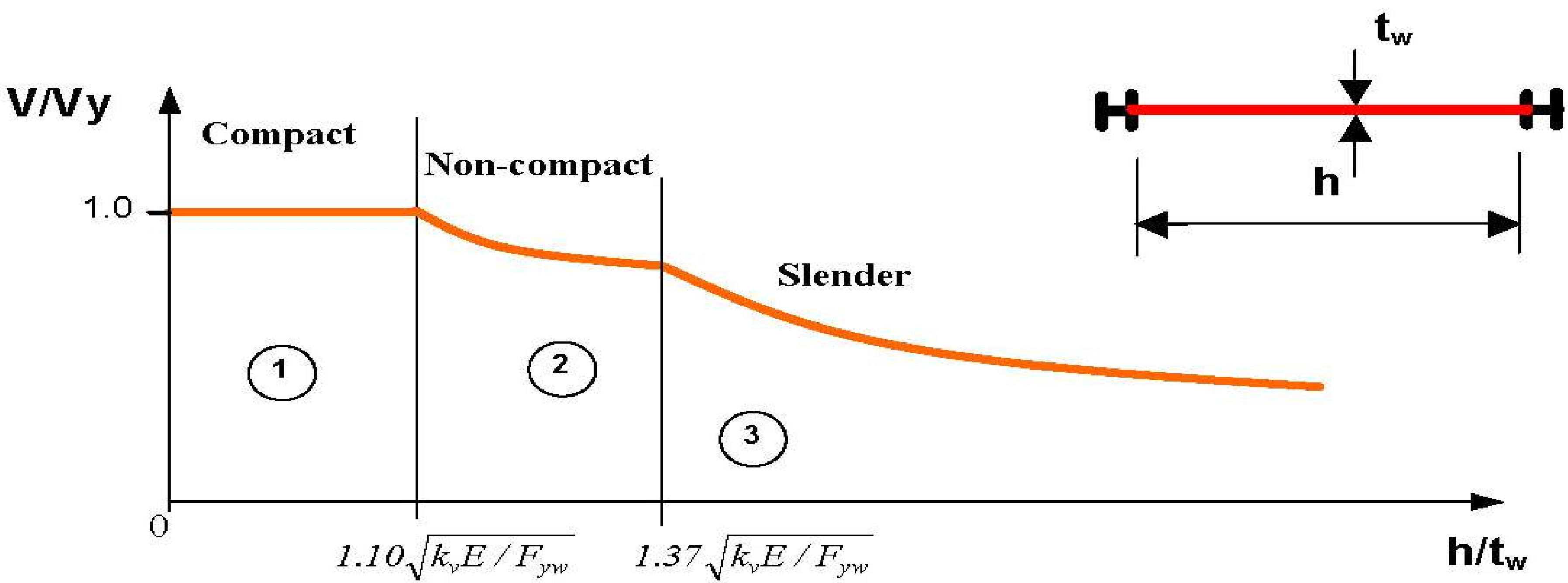
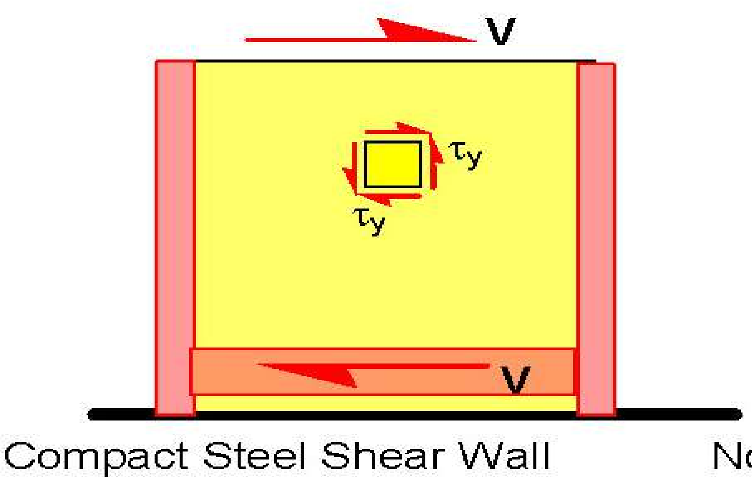
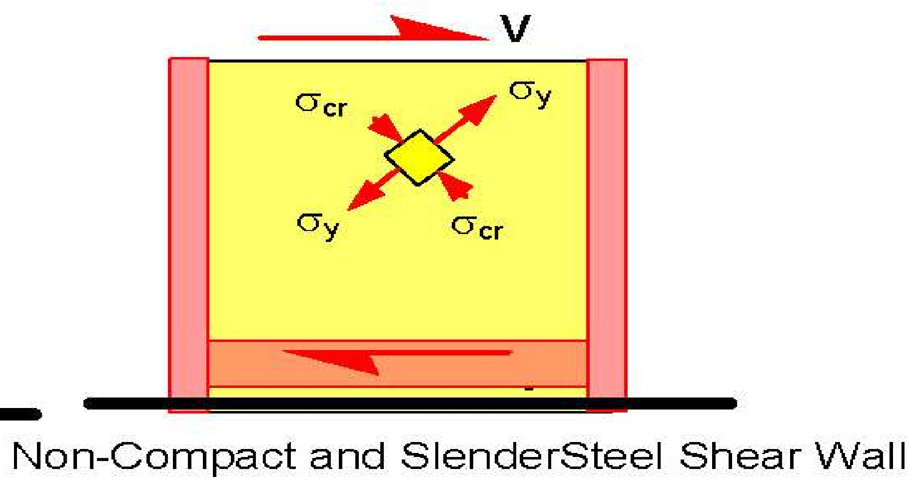


Figure 5.2. Three regions of behavior of steel shear walls





The shear capacity of steel plate shear walls, in LRFD format, $\phi_v V_n$, where $\phi_v = 0.90$ and V_n is determined as follows:

A. For compact shear walls when $h/t_w \leq 1.10\sqrt{k_v E / F_{yw}}$

$$V_n = 0.6 A_w F_{yw} \tag{5.1}$$

B. For non-compact and slender shear walls when $h/t_w > 1.10\sqrt{k_v E / F_{yw}}$

$$V_n = 0.6 A_w F_{yw} \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \tag{5.2}$$

Where k_v is given by:

$$k_v = 5 + \frac{5}{(a/h)^2} \tag{5.3}$$

The value of k_v should be taken as 5.0 if a/h is greater than 3.0 or $[260/(h/t_w)]^2$. The value of C_v is given by AISC (1999) as:

(a) For $1.10\sqrt{\frac{k_v E}{F_{yw}}} \le \frac{h}{t_w} \le 1.37\sqrt{\frac{k_v E}{F_{yw}}}$:

$$C_v = \frac{1.10\sqrt{k_v E / F_{yw}}}{h / t_w} \tag{5.4}$$

(b) For $\frac{h}{t_w} > 1.37\sqrt{\frac{k_v E}{F_{yw}}}$:

$$C_v = \frac{1.51 k_v E}{(h / t_w)^2 F_{yw}} \tag{5.5}$$

In addition, the boundary beams and columns of shear walls should satisfy the following b/t requirements given by the AISC Seismic Provisions (AISC, 1997):

$$b_f/2t_f \leq 52/\sqrt{F_y} \tag{5.10}$$

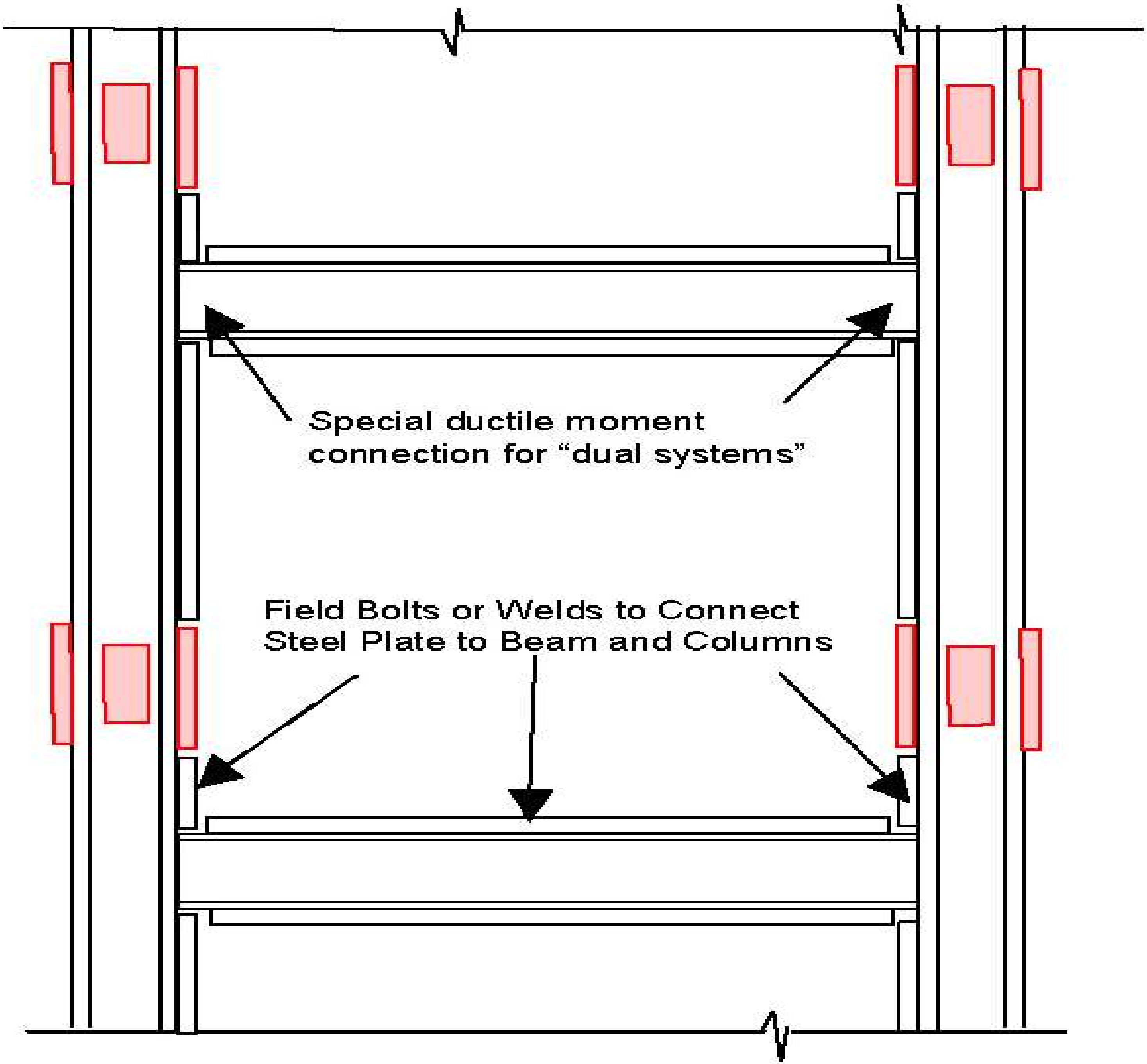
The above equation in non-dimensional form can be written as:

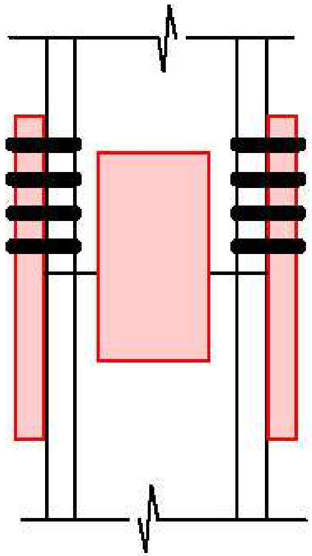
$$b_f/2t_f \leq 0.31/\sqrt{E/F_y} \tag{5.10a}$$

$$\mathrm{h_c/t_w} \leq 520/\sqrt{\mathrm{F_y}} \tag{5.11}$$

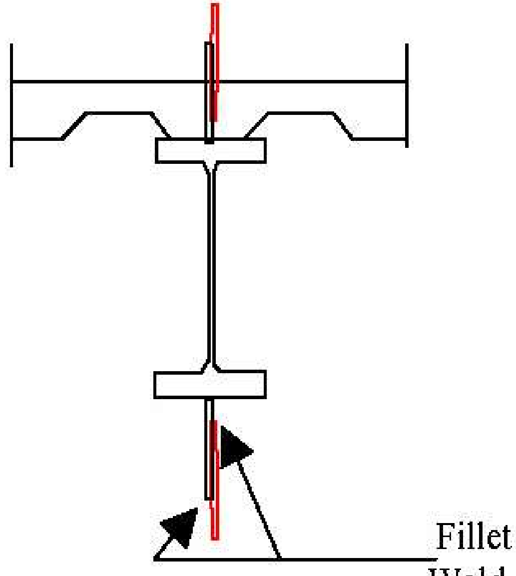
The above equation in non-dimensional form can be written as:

$$h_c/t_w \leq 3.10/\sqrt{E/F_y} \tag{5.11a}$$

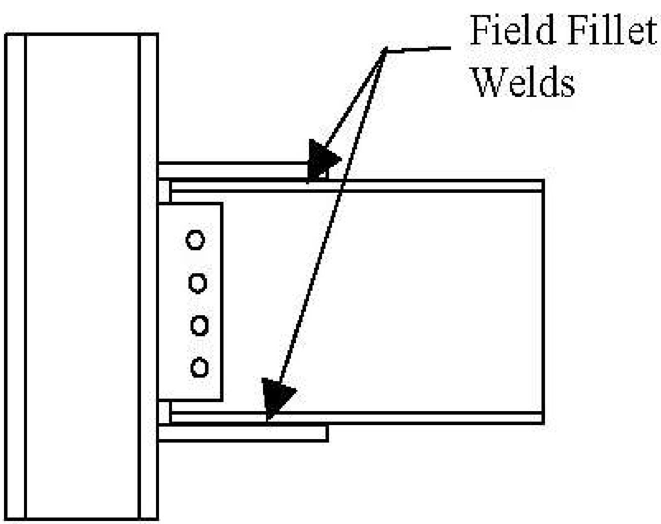

Special ductile moment
connection for “dual systems”
Field Bolts or Welds to Connect
Steel Plate to Beam and Columns



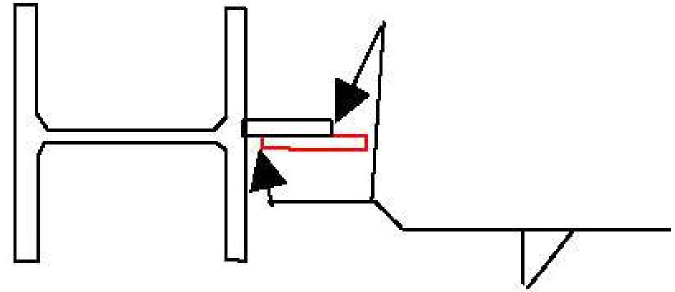
Column Splice



Beam Section



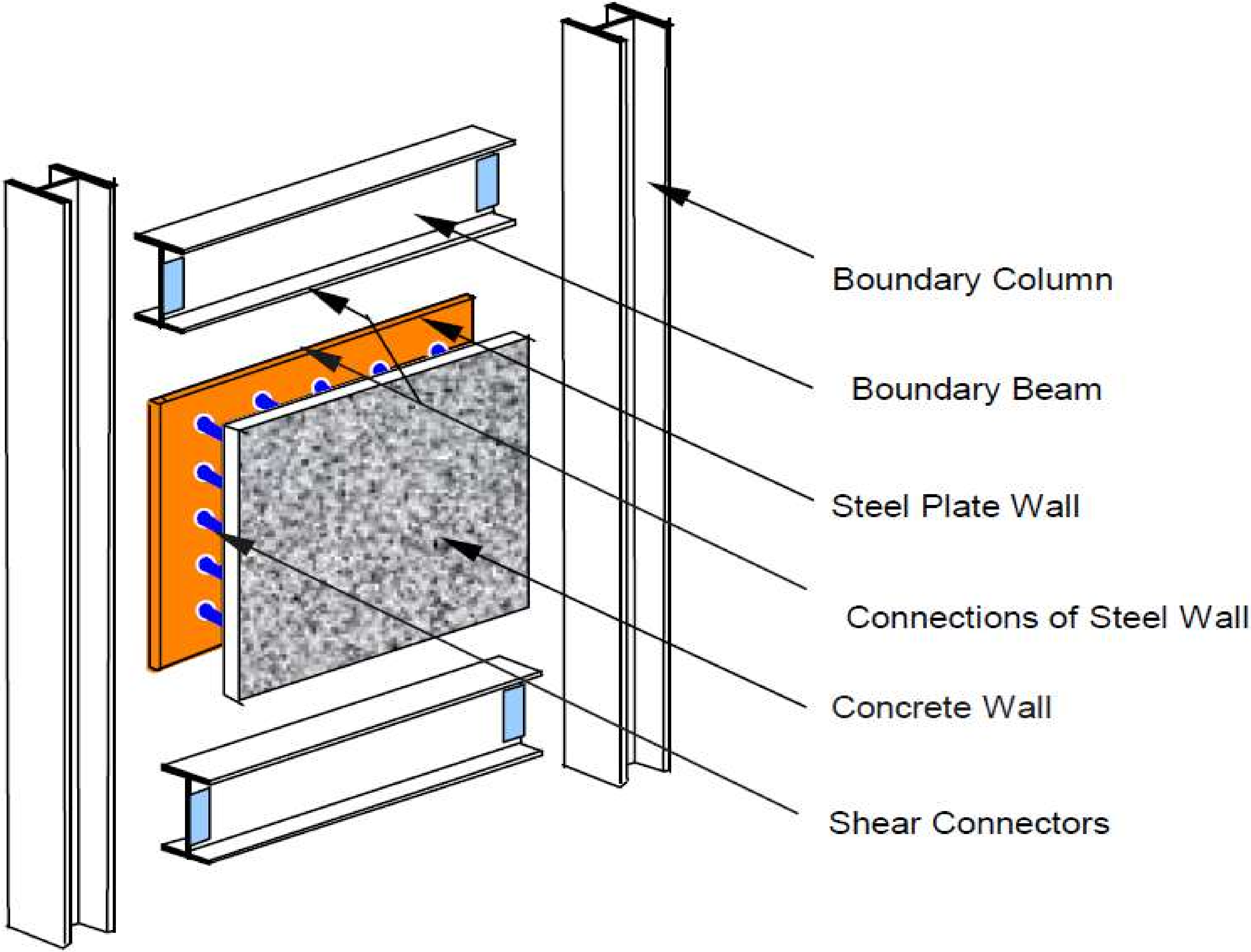
Beam-to-Column Moment Connection



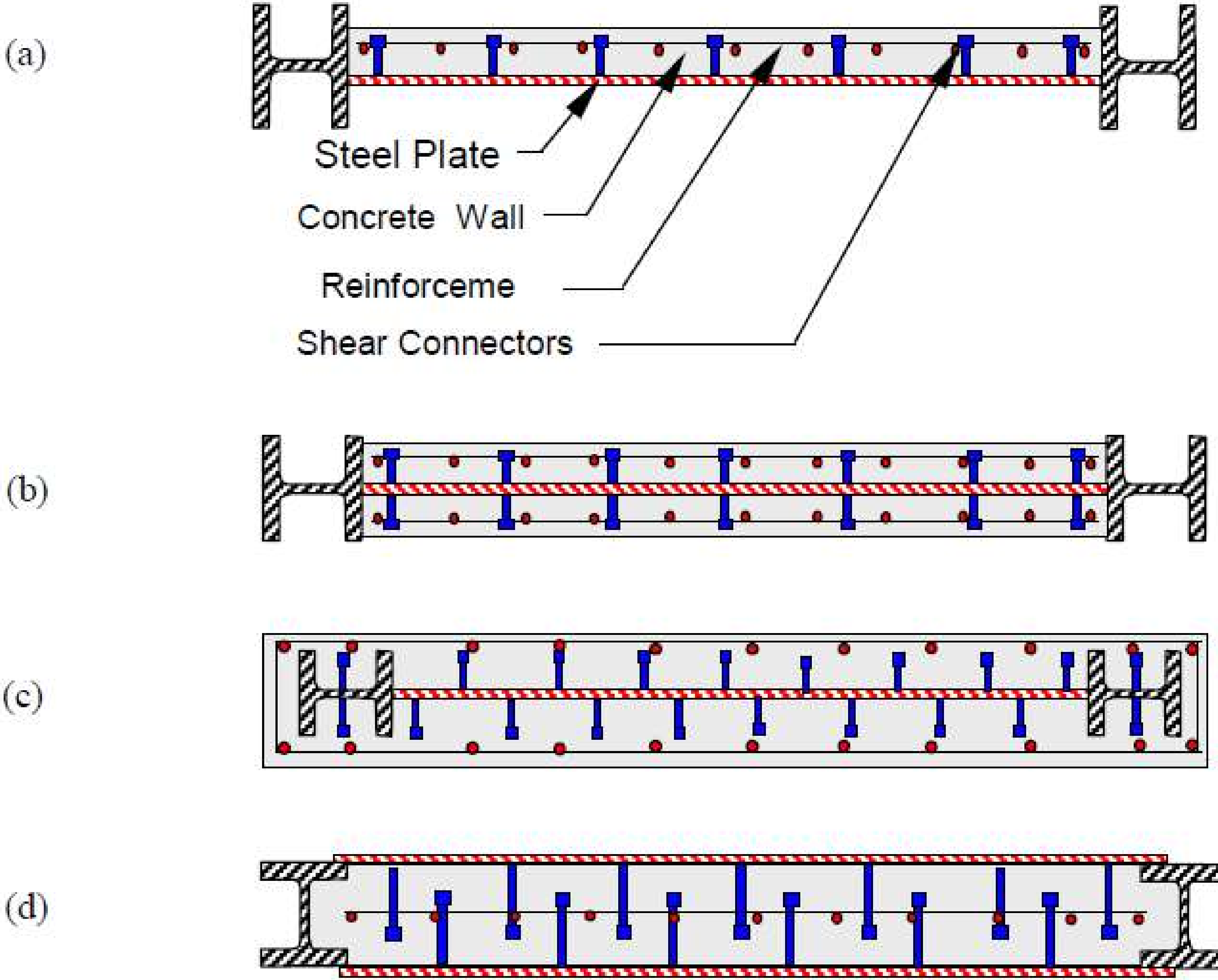
Column Section

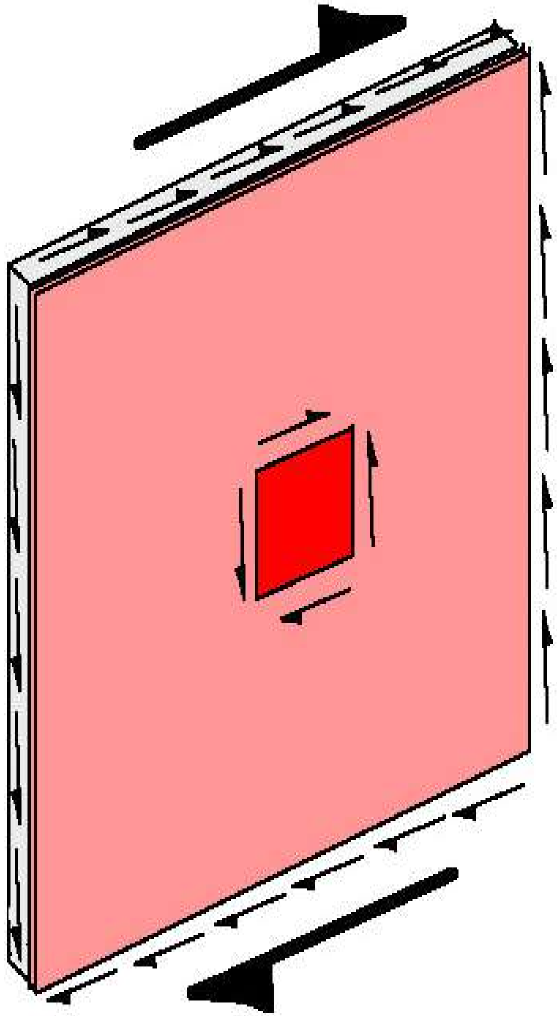
COMPOSITE PLATE SHEAR WALLS (C-PSW) AISC-341

- **1. Scope**
- *Composite plate shear walls (C-PSW) shall be designed in conformance with this section. Composite plate shear walls consist of **steel plates with reinforced concrete encasement on one or both sides of the plate, or steel plates on both sides of reinforced concrete infill,** and structural steel or composite boundary members*
- **2. Basis of Design**
- *C-PSW designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through yielding in the plate webs. **The horizontal boundary elements (HBE) and vertical boundary elements (VBE) adjacent to the composite webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded steel webs along with the reinforced concrete webs after the steel web has fully yielded, except that plastic hinging at the ends of HBE is permitted.***

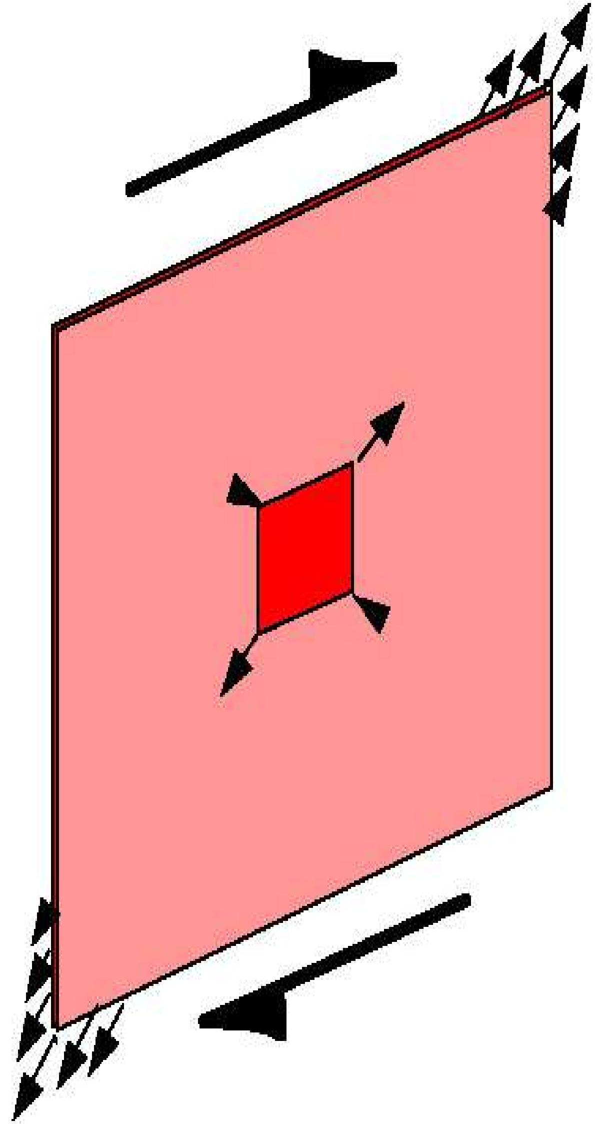


Boundary Column
Boundary Beam
Steel Plate Wall
Connections of Steel Wall
Concrete Wall
Shear Connectors

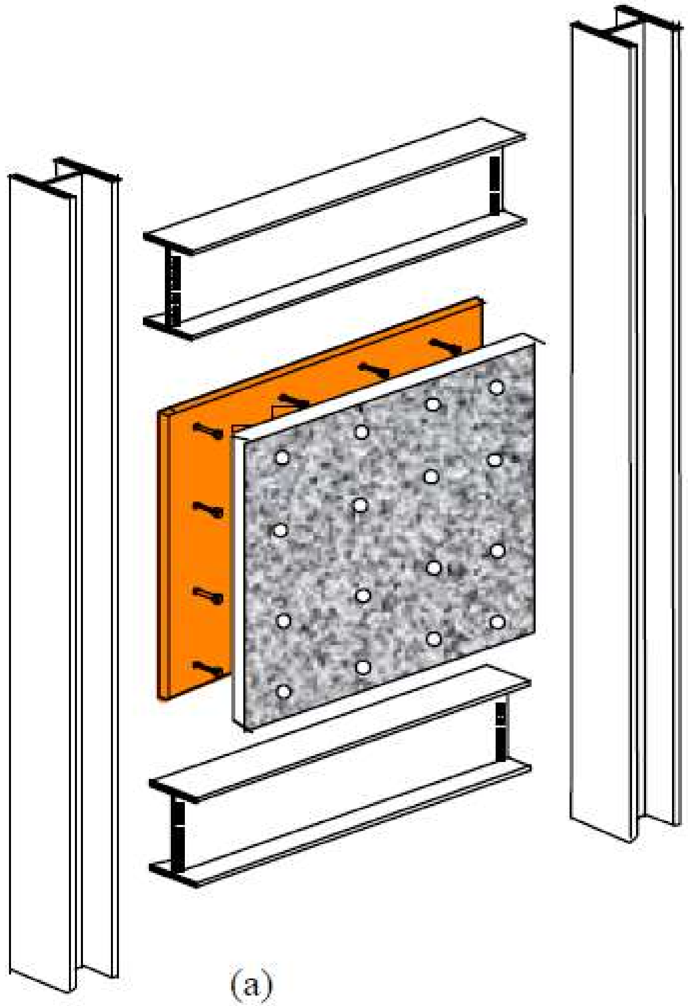

(a)
Steel Plate
Concrete Wall
Reinforceme
Shear Connectors
(b)
(c)
(d)



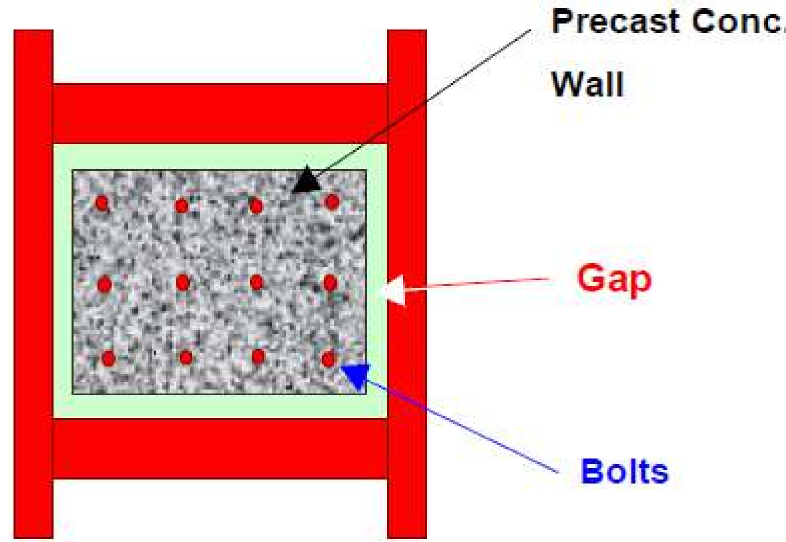
a. Shear Wall Elements Under Pure Shear



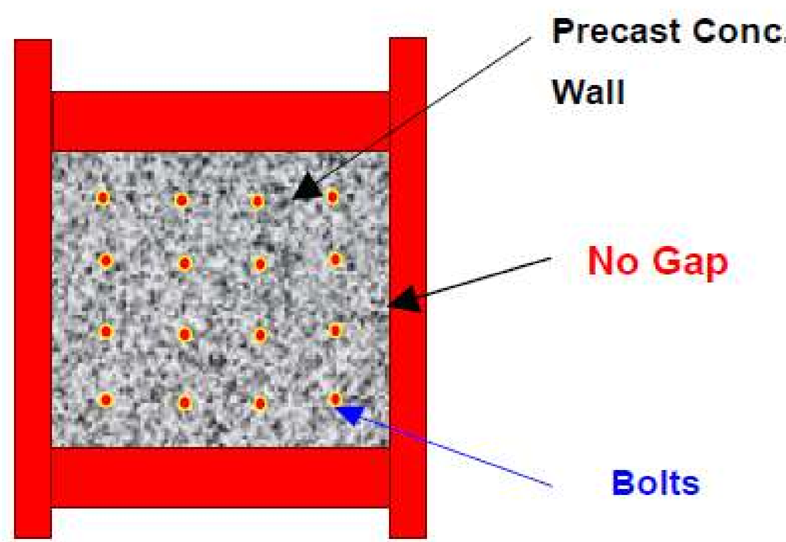
b. Shear Wall Elements Under Tension Field Action



(a) Composite Shear Wall Studied



(b) Innovative Composite Wall



(c) Traditional Composite Wall

Note: Steel shear wall is fillet-welded to steel tab plates on all four boundaries. The tab plates are fillet-welded to the boundary beam and column flanges.

- **3. Analysis**
- **3a. Webs**
- *Steel webs shall be designed to resist the seismic load, E, determined from the analysis required by the applicable building code. The analysis shall account for openings in the web.*
- **3b. Other Members and Connections**
- ***Columns, beams and connections in C-PSW shall be designed to resist seismic forces determined from an analysis that includes the expected strength of the steel webs in shear, $0.6R_yF_yA_{sp}$, and any reinforced concrete portions of the wall active at the design story drift****. The vertical boundary elements (VBE) are permitted to yield at the base.*
- **4. System Requirements**
- **4a. Steel Plate Thickness**
- *Steel plates* ***with thickness less than 3/8 in. (9.5 mm)*** *are not permitted*

- **4b. Stiffness of Vertical Boundary Elements**
- *The VBE shall satisfy the requirements of Section F5.4a.*
- **4c. HBE-to-VBE Connection Moment Ratio**
- *The beam-column moment ratio shall satisfy the requirements of Section F5.4b.*
- **4d. Bracing**
- *The bracing shall satisfy the requirements of Section F5.4c.*
- **4e. Openings in Webs**
- *Boundary members shall be provided around openings in shear wall webs as required by analysis.*
- **5. Members**
- **5a. Basic Requirements**
- *Steel and composite HBE and VBE shall satisfy the requirements of Section D1.1 for highly ductile members*

- **5b. Webs**
- The *design shear strength,* φV_n*, or the allowable shear strength ,*V_n/Ω*, for the limit* state of shear yielding with a composite plate conforming to Section H6.5c shall be taken as:
- $V_n = 0.6A_{sp}F_y$ *(H6-1)*
- $\varphi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
- where
- A_{sp} *= horizontal area of stiffened steel plate, in.*^{2} *(mm*^{2}*)*
- F_y *= specified minimum yield stress of the plate, ksi (MPa)*
- V_n *= nominal shear strength of the steel plate, kips (N)*
- *The available shear strength of C-PSW with a plate that does not meet the stiffening requirements in Section H6.5c shall be based upon the strength of the plate as given in Section F5.5 and satisfy the requirements of Specification Sections G2 and G3.*

- **5c. Concrete Stiffening Elements**
- *The steel plate shall be adequately stiffened by encasement or attachment to a reinforced concrete panel. Conformance to this requirement shall be demonstrated with an elastic plate buckling analysis showing that the composite wall can resist a nominal shear force equal to V_{ns}.*
- ***The concrete thickness shall be a minimum of 4 in. (100 mm) on each side when concrete is provided on both sides of the steel plate and 8 in. (200 mm) when concrete is provided on one side of the steel plate.*** *Steel headed stud anchors or other mechanical connectors shall be provided to prevent local buckling and separation of the plate and reinforced concrete. Horizontal and vertical reinforcement shall be provided in the concrete encasement to meet or exceed the requirements in ACI 318 Section 14.3.* ***The reinforcement ratio in both directions shall not be less than 0.0025. The maximum spacing between bars shall not exceed 18 in. (450 mm).***

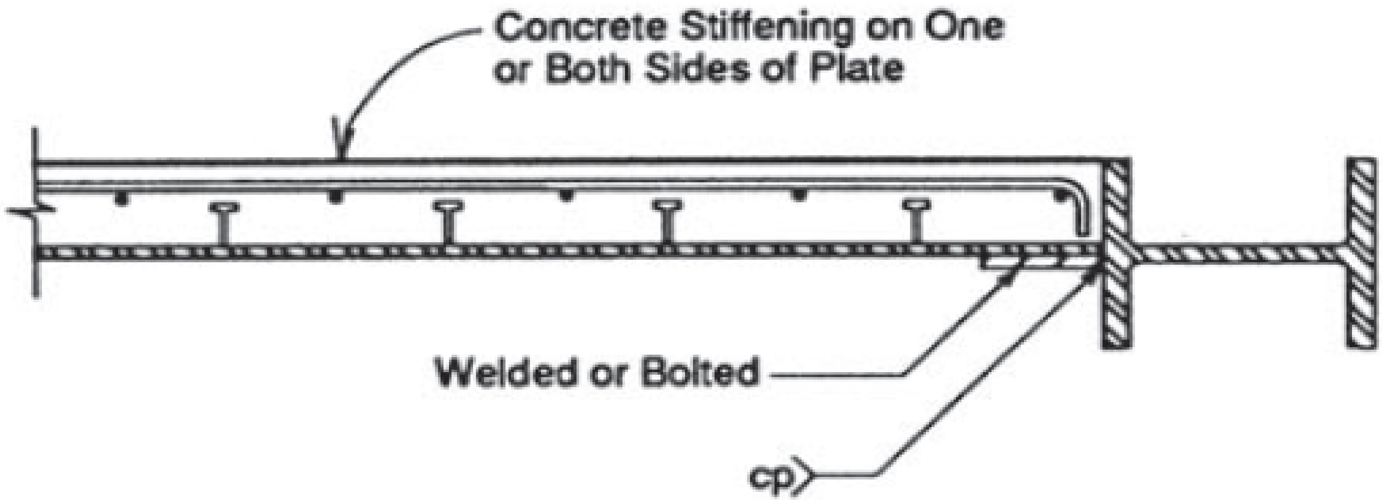


Fig. C-H6.1. Concrete stiffened steel shear wall with steel boundary member.

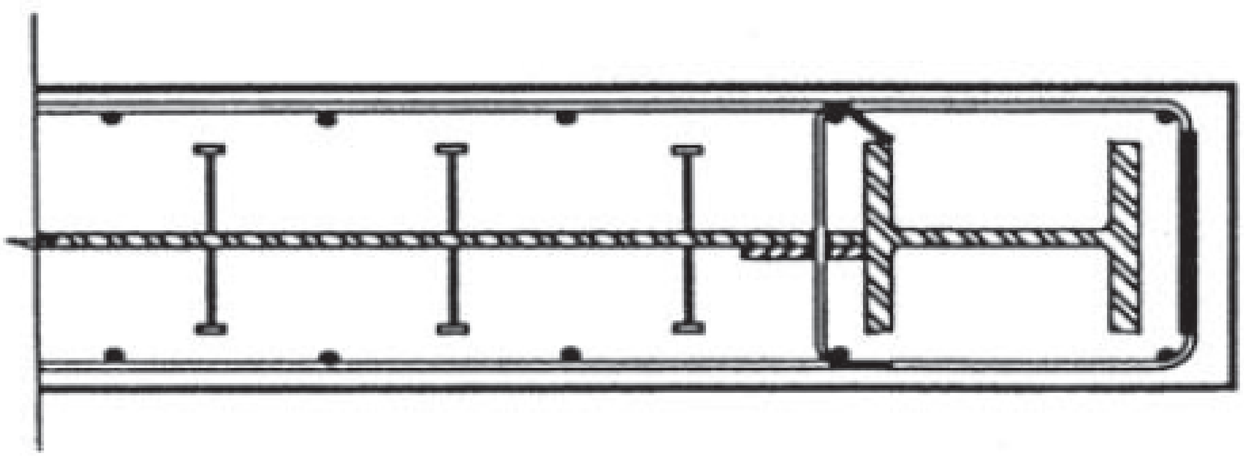


Fig. C-H6.2. Concrete stiffened steel shear wall with composite (encased) boundary member.

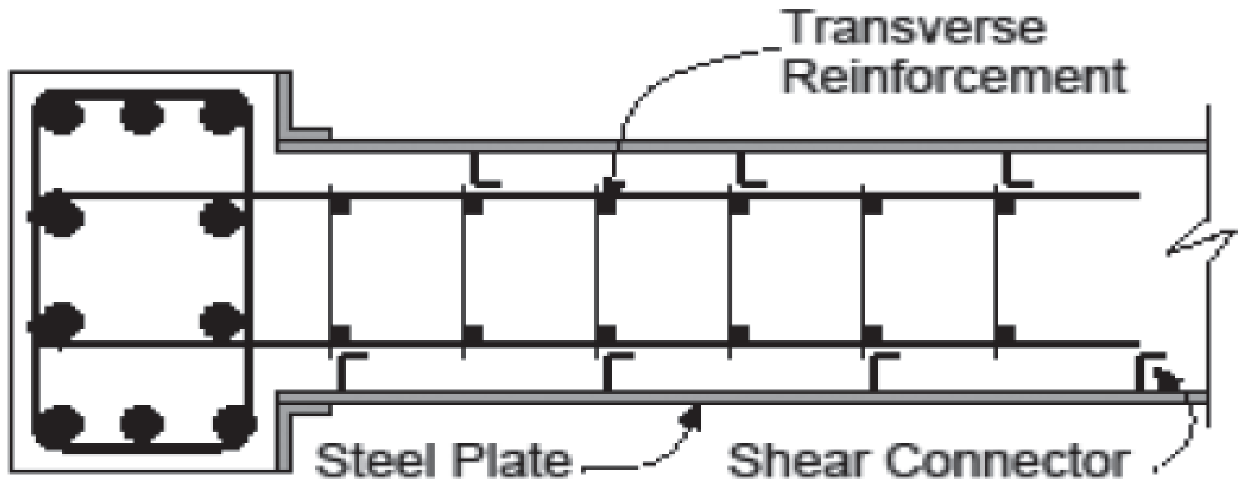


Fig. C-H6.3. Concrete filled C-PSW with a boundary element and transverse reinforcement.

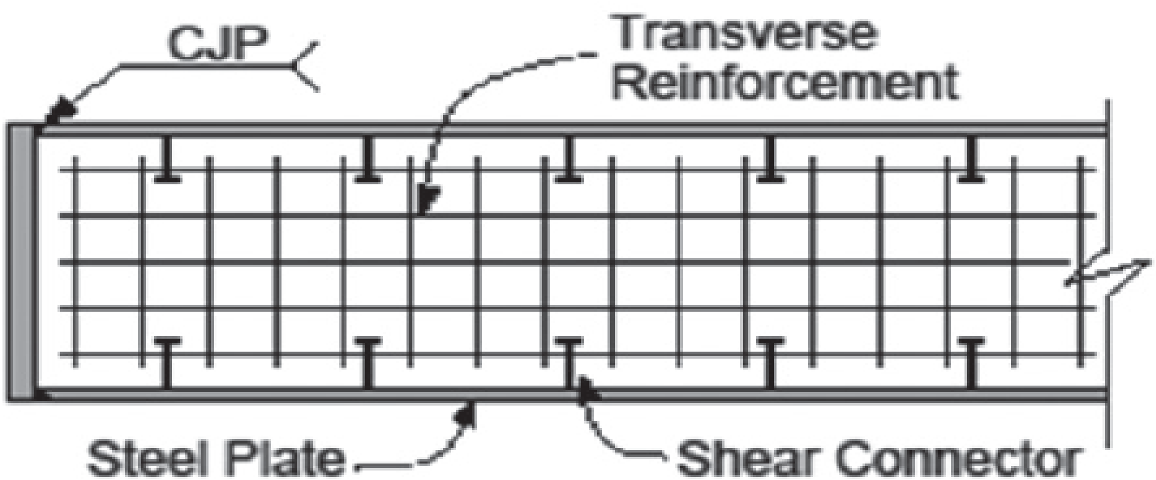


Fig. C-H6.4. Concrete filled C-PSW with transverse reinforcement to provide integrity of the concrete infill.

- **5d. Boundary Members**
- *Structural steel and composite boundary members shall be designed to resist the expected shear strength of steel plate and any reinforced concrete portions of the wall active at the design story drift. Composite and reinforced concrete boundary members shall also satisfy the requirements of Section H5.5b. Steel boundary members shall also satisfy the requirements of Section F5.*
- **5e. Protected Zones**
- *There are no designated protected zones.*
- **6. Connections**
- **6a. Demand Critical Welds**
- *The following welds are demand critical welds, and shall satisfy the requirements of Section A3.4b and I2.3:*
- ***(1) Groove welds at column splices***
- ***(2) Welds at the column-to-base plate connections***

- *Exception: Where it can be shown that column hinging at, or near, the base plate is precluded by conditions of restraint, and in the absence of net tension under load combinations including the amplified seismic load, demand critical welds are not required.*
- *(3) Welds at HBE-to-VBE connections*
- **6b. HBE-to-VBE Connections**
- *HBE-to-VBE connections shall satisfy the requirements of Section F5.6b.*
- **6c. Connections of Steel Plate to Boundary Elements**
- The steel plate shall be continuously welded or bolted on all edges to the structural steel framing and/or steel boundary members, or the steel component of the composite boundary members. **Welds and/or slip-critical high-strength bolts required to develop the nominal shear strength of the plate shall be provided.**

- **6d. Connections of Steel Plate to Reinforced Concrete Panel**
- *The steel anchors between the steel plate and the reinforced concrete panel shall be designed to prevent its overall buckling. Steel anchors shall be designed to satisfy the following conditions:*
- **(1) Tension in the Connector**
- *The steel anchor shall be designed to resist the tension force resulting from inelastic local buckling of the steel plate.*
- **(2) Shear in the Connector**
- *The steel anchors collectively shall be designed to transfer the expected strength in shear of the steel plate or reinforced concrete panel, whichever is smaller.*
- **6e. Column Splices**
- *Column splices shall be designed following the requirements of Section G2.6f*

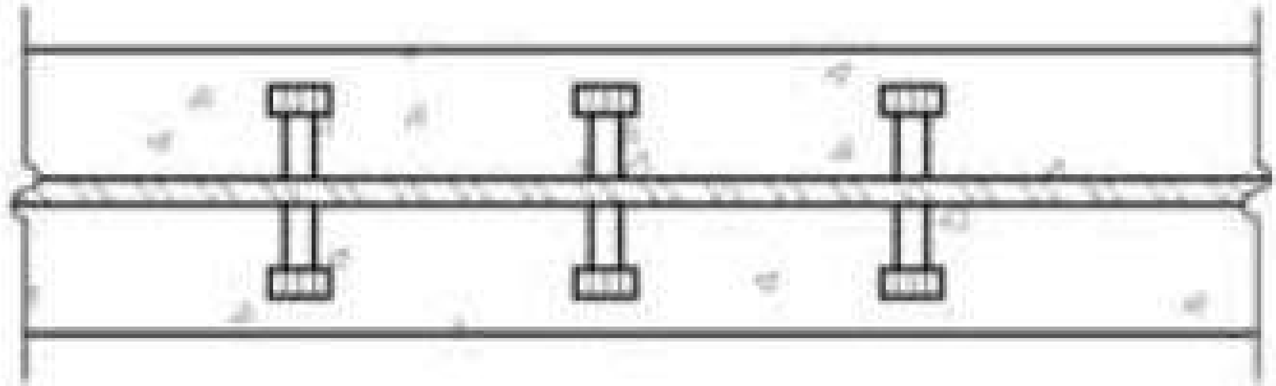


Fig. 3–8. C-SPW with concrete on both sides of the web plate.

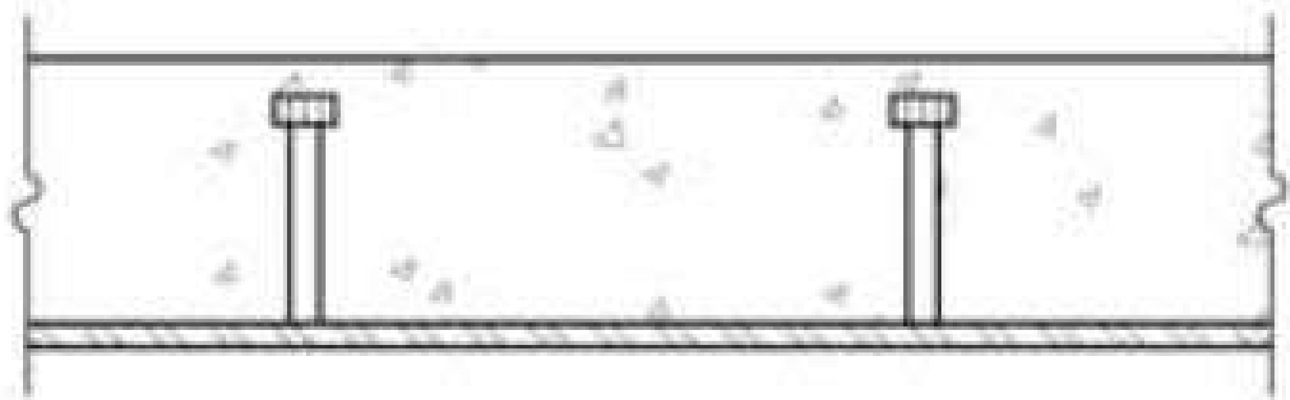
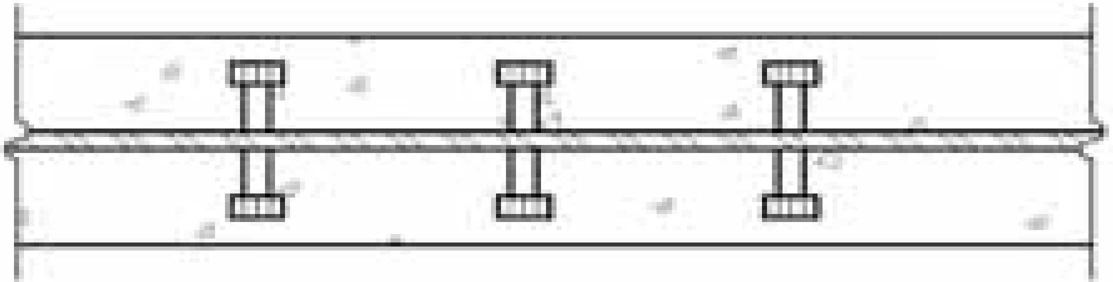


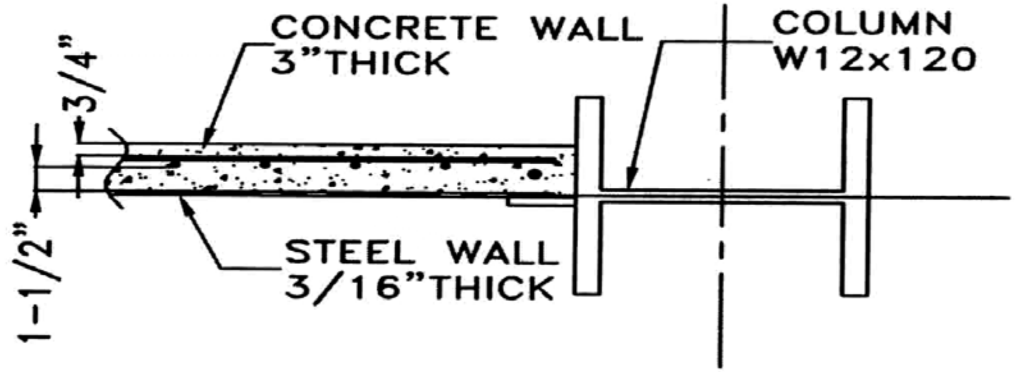
Fig. 3–9. C-SPW with concrete on one side of the web plate.



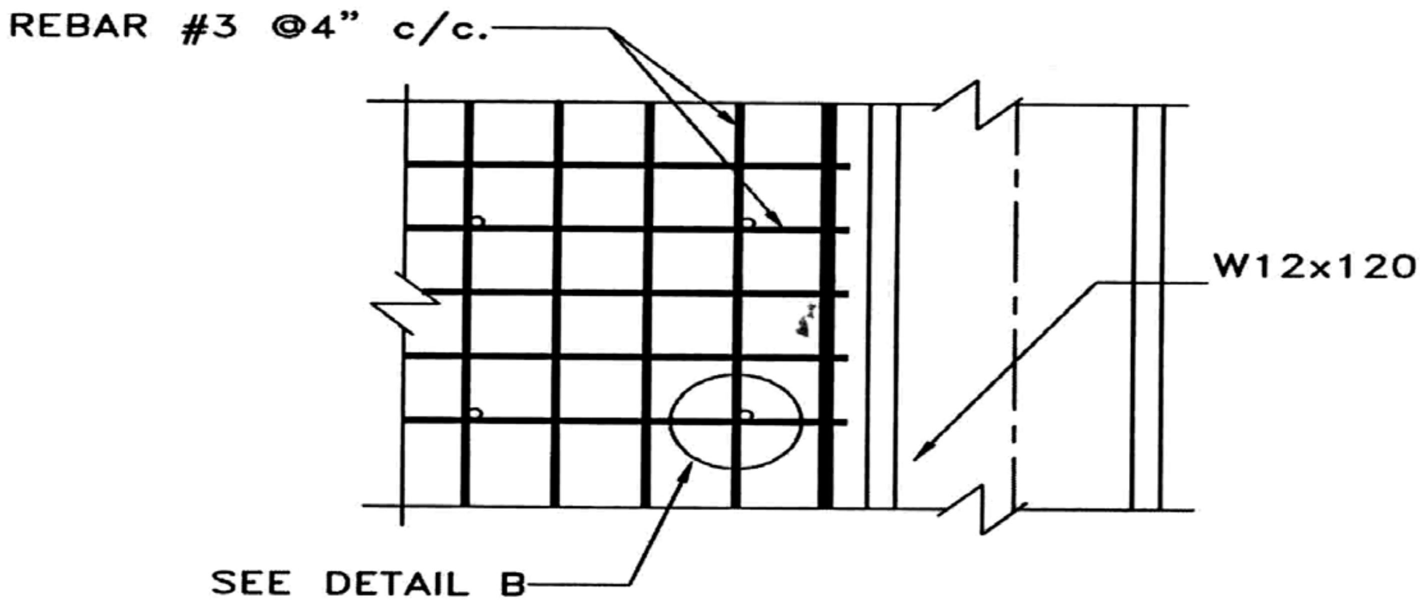
(a)

(b)

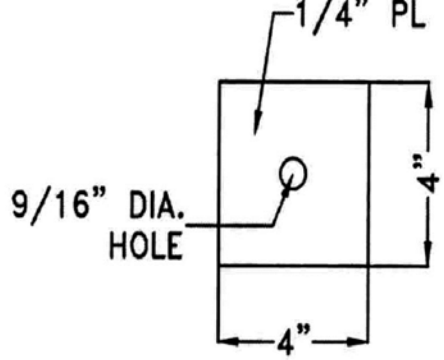
(c)



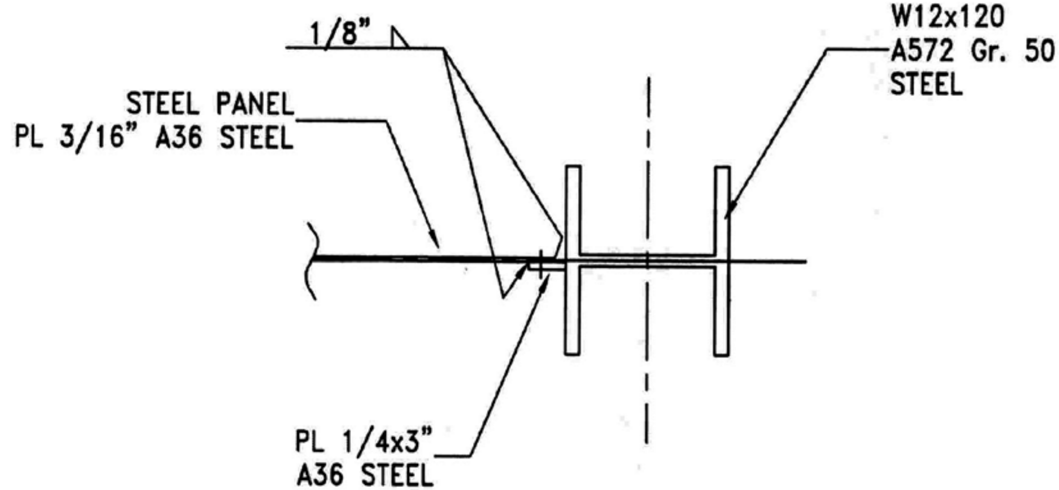
PLAN VIEW



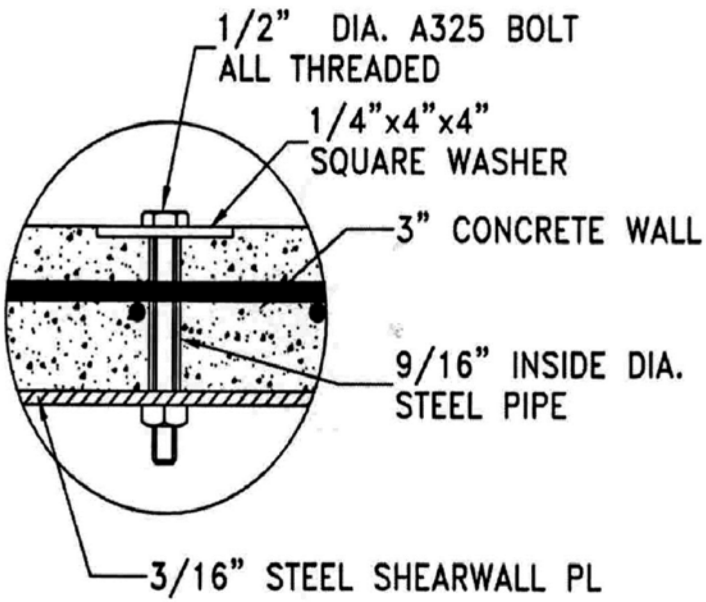
FRONT ELEVATION



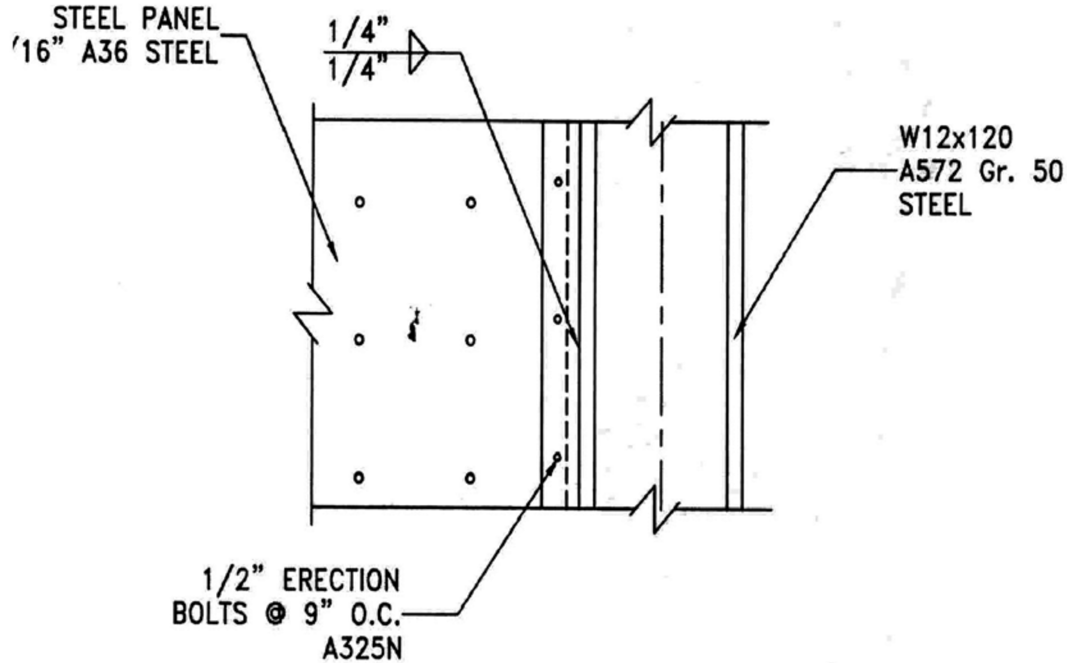
WASHER PLATE



PLAN VIEW



DETAIL B



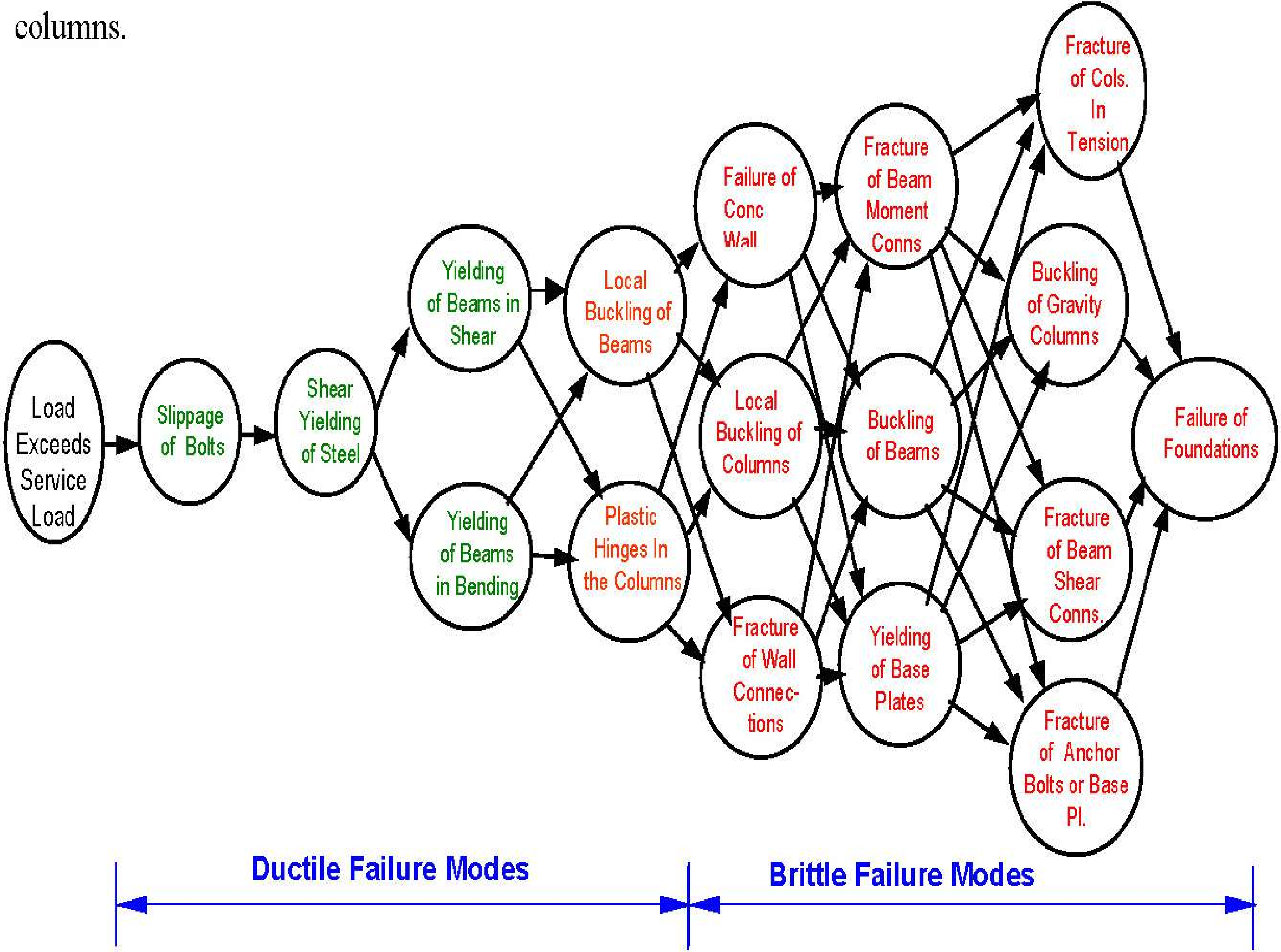
ELEVATION

- *Major failure modes*
- *Failure modes of composite shear walls*
- *1. Slippage of bolts (ductile).*
- *2. Yielding of the steel plate (ductile).*
- *3. Buckling of the steel plate (ductile).*
- *4. Cracking and spalling of the concrete wall (ductile/brittle)*
- *5. Fracture of the shear connectors (brittle)*
- *6. Fracture of the wall plate (brittle).*
- *7. Fracture of the connections of steel wall to boundary columns and beams (brittle).*

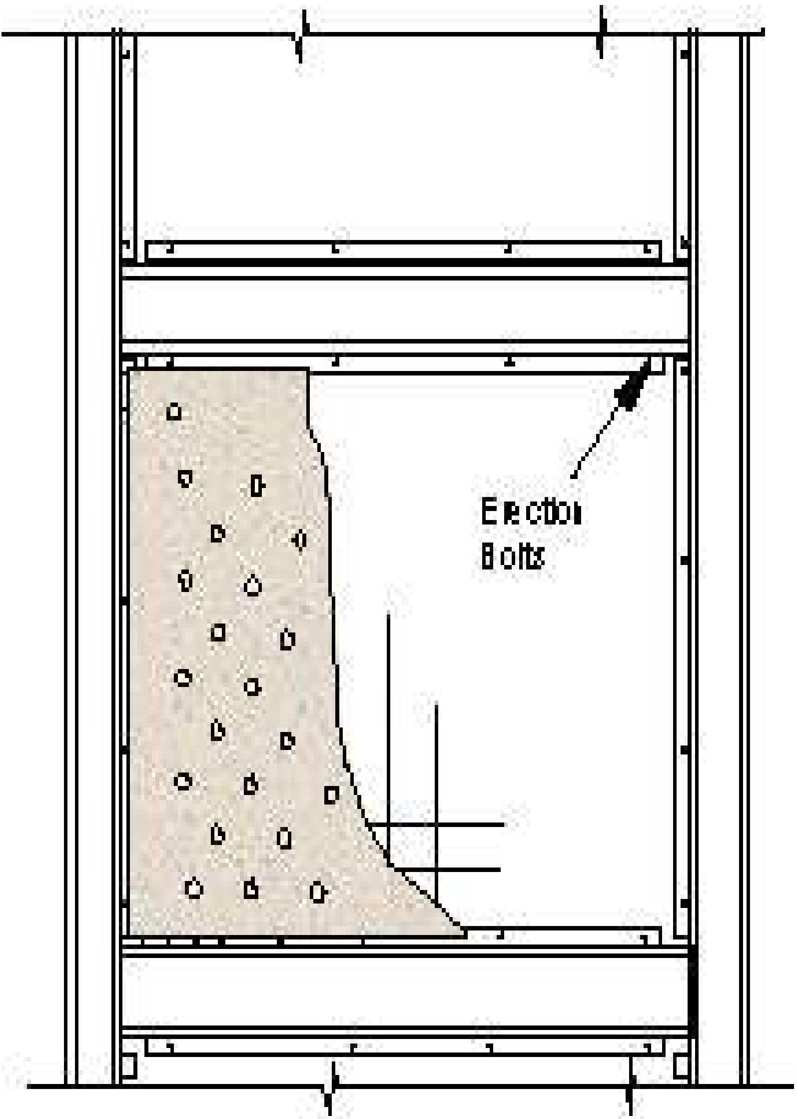
- *Failure modes of top and bottom beams*
- *8. Shear yielding of the top and bottom beams (ductile).*
- *9. Plastic hinge formation in the top and bottom beams (ductile).*
- *10. Local buckling in the top and bottom beam flanges or web (ductile if $b/t \leq \lambda_p$).*
- *11. Fracture in beam-to-column moment connections (brittle).*
- *12. Overall or lateral-torsional buckling of beams (brittle).*
- *13. Fracture of shear connections of beams (brittle).*

- *Failure modes of Boundary Columns*
- *14. Plastic hinge formation at the top and bottom of column(ductile).*
- *15. Local buckling of boundary columns (ductile if $b/t \leq \lambda p$).*
- *16. Overall buckling of boundary columns (ductile if $\lambda c=(KL/\pi r)\sqrt{(Fy / E)} \leq 1.0$.)*
- *17. Yielding of base plates of boundary columns in uplift (ductile)*
- *18. Tension fracture of boundary columns or their splices (brittle).*
- *19. Fracture of anchor bolts or base plates at the base of the columns in uplift (brittle)*
- *20. Fracture of the column base plates in bending and/or uplift (brittle)*
- *21. Failure of the foundations of the wall (brittle).*

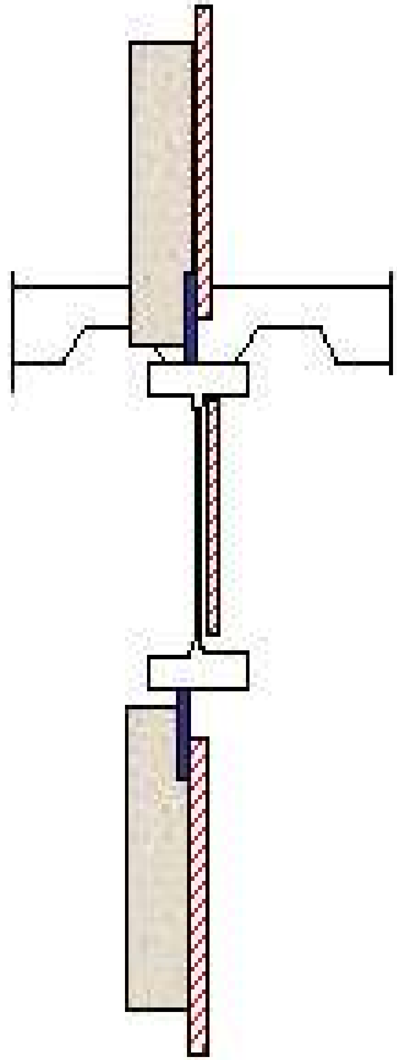
columns.



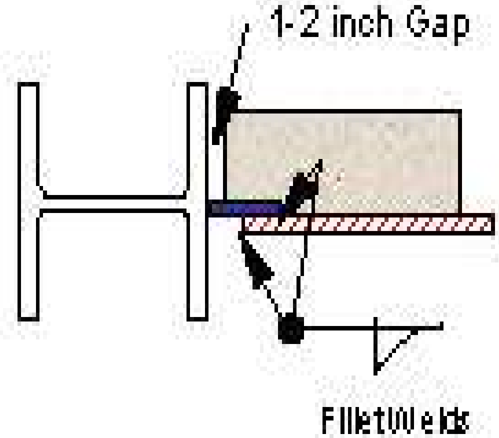
Load Exceeds Service Load
Slippage of Bolts
Shear Yielding of Steel
Yielding of Beams in Shear
Yielding of Beams in Bending
Local Buckling of Beams
Plastic Hinges In the Columns
Failure of Conc Wall
Local Buckling of Columns
Fracture of Wall Connec-tions
Fracture of Beam Moment Conns
Buckling of Beams
Yielding of Base Plates
Fracture of Cols. In Tension
Buckling of Gravity Columns
Fracture of Beam Shear Conns.
Fracture of Anchor Bolts or Base Pl.
Failure of Foundations
Ductile Failure Modes
Brittle Failure Modes



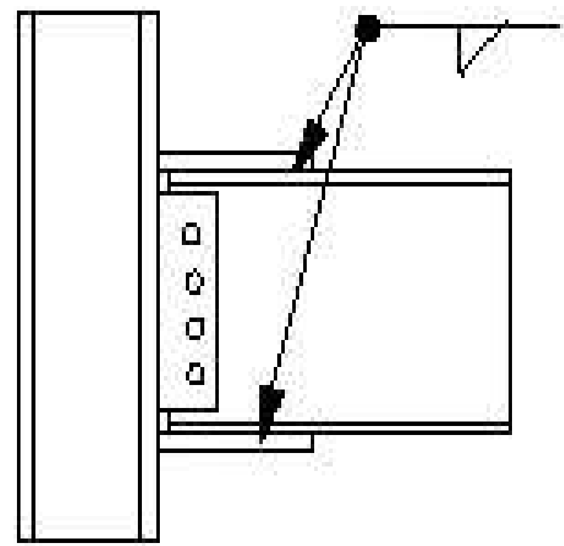
Elevation



Beam Section



Column Section



Beam-to-Column Moment Connection